ENGINEERING RECORD

A WEEKLY JOURNAL DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

Published by the McGraw Publishing Company, Inc.

Volume 68 NEW YORK, SATURDAY, DECEMBER 13, 1913 Number 24

Death of Colonel Gaillard The death of Lieut.-Col. David DuB. Gaillard in Baltimore last week marks the first break in the ranks of those great engineers to whom the Panama achievement is to be credited. Not less bravely than the soldier whose career ends on the battlefield did he give up his life in the service of his country. The severity of his duties combined with the climate undermined his health. As division engineer in charge of all of the excavation between the Gatun Dam and the Pedro Miguel locks he bore a very heavy responsibility in the execution of the great enterprise. His work and ability are testified to by all of his associates, and none of the praise that is justly his is more sincere than that of the chief engineer of the canal himself. It is entirely fitting that Congress should show appreciation of his efforts by special enactment in favor of his widow, as has been proposed. With the other men still fighting the engineering battles on the Isthmus, his name will go down in history, for the Panama Canal will assume important rank in American annals.

Camp Sanitation At the recent Conservation Congress Miss Mabel Boardman, chairman of the executive committee of the American Red Cross, pointed out the work that lay before sanitarians in lumber camps. The work is by no means a small one. In the State of Washington alone no fewer than 47,000 men are employed in the lumber districts, while 800,000 is the estimate for the whole country. These men all live under comparatively primitive conditions, in more or less temporary camps, where sanitation, unless the management is progressive, is likely to be one of the last thoughts. She offered, on behalf of the Red Cross, to co-operate with lumber companies in supplying physicians who would give simple instruction not only as to camp sanitation but also as to first aid and the prevention of accidents. Attention is called to the matter here because good results would probably be obtained by investing the State board of health with supervisory and police powers. This has worked successfully in at least one State. The subject deserves the attention of sanitarians throughout the country.

Is the Old Bridge Safe? Investigations of the adequacy of old bridges under increased traffic stresses are quite general in railroad offices, but even so the subject holds possibility of so much trouble that a sermon preached upon such a forceful text is justified. It is a human trait to "weary in well doing." Recently a condition of affairs was discovered in an old bridge which should cause the same tightening of discipline as would a failure of the structure. The bridge was a deck cantilever with a maximum span of about 330 ft., put in service in 1885. After twenty-seven years of service it was declared too light for the heavier rolling stock and was replaced by a new structure. The old bridge did not show any considerable amount of deflection under the heavy loading, but after being dismembered it was found that the tension members had become so badly crystallized that, in the opinion of the engineer in charge of the work, "it was a wonder that the bridge stood up." The fact that the deck of this structure was about 100 ft. above a stream so swift that soundings had never been possible and that the depth of the canyon was unknown shows how serious failure at this point would have been. If the bridge had failed one can readily imagine the queries that would have passed around the engineering department of railroads as to the possibility of similar conditions obtaining on their structures. Should not the incident serve as a sufficient warning for the same sort of rigid inquiry that would have resulted had this bridge gone into the unfathomable canyon?

Dragging Down the Profession "What is a progressive?" is a question that can be asked as well in engineering to-day as in politics. Engineers of the highest standing are endeavoring through national and local societies to elevate the standards of the profession and thus secure for the engineer the recognition which his training and responsibility warrant. In Chicago a number of engineers have banded together as the Chicago Technical League for the purpose of securing larger compensation, and they undoubtedly feel that they, too, are raising the standing of engineers, for their standard is based on remuneration. They have thought to strengthen themselves by affiliation with the American Federation of Labor. They unquestionably regard themselves as progressives, though the majority of the profession would prefer to accord that title to those who are working to elevate the standard in another direction. The great upward effort in engineering has been to establish itself as one of the recognized professions. The gentlemen in Chicago evidently are not in sympathy with that movement, for they have deliberately cast their lot with the skilled mechanic, the class to which the engineer was at one time supposed to belong. Instead of elevating they are pulling down; instead of securing professional recognition they have gone back generations and would some day require that the engineers again fight their way upward. That they will find their new association uncongenial—not in a personal way—but from the standpoint of aims and occupation the Engineering Record unhesitatingly predicts. Of course, the political possibilities of such an organization with certain types of politicians may be large, and here again would come the claim of the Chicago Technical League that it was working in the same direction as the leaders in engineering—for has not the American Institute of Consulting Engineers urged upon the Governor of New York, the Mayor of New York City and the President of the United States the appointment of engineers to high places in which their technical knowledge would be invaluable to the respective governments? The Engineering Record believes, however, that the endorsement of the Chicago Technical League—which means the endorsement of the American Federation of Labor—will not go far, except with politicians of the old spoils type. When the qualifications of an engineer are at issue, the opinion of engineers, not of laboring men, will be sought, no matter how excellent these men may be personally. If a labor appointment were at stake, surely the endorsement of a national engineering society would have no weight. The proposed plan of the Chicago Technical League to endorse an engineer for membership on the new Illinois Public Utility Commission will, therefore, be watched with interest. The Engineering Record hopes that the members of the Chicago Technical League will see the error of their methods and discontinue their relations with the American Federation of Labor.

Design of Rolling Dams Attention was called in these columns on Nov. 22 to the advantages of the roller type of movable dam. The subject holds such possibilities for American practice that the Engineering Record presents on page 654 a comprehensive article on the subject, embodying not only the principal details with which a designer needs to be acquainted but also data on methods of calculation. That this type of dam promises to come into rapid favor in the United States is evidenced by the recent installation on the Boise project of the Reclamation Service and the adoption of a goodly sized structure for the Grand River Dam in Colorado for the same service and another for the Long Lake Development of the Washington Water Power Company. That there is need for data such as are presented in the paper in this issue is apparent from a recent case where a dam of the roller type had not been considered at all in a location particularly adapted to this type of structure. One point should be especially noted—that the depth of opening is not limited by the economical height of a single roller. Although spans with two or more rollers have not been built there seems to be no good reason why the type could not be employed. The limitations will enter, of course, in the dimensions and height of piers, but it seems that these would not be reached even with two rollers. The rolling dam, naturally, is more expensive per linear foot than other types, but the fact that it permits deeper and larger discharge openings shortens very considerably the spillway section of the dam. This point was made clear in the editorial in the issue of Nov. 22. Of course, the higher cost must be balanced against the saving in concrete and excavation due to the shortening of the spillway section, but this is only one of the ordinary factors entering in the comparison of any engineering structure. It is not out of place to call attention to the fact that the leakage can be cut down to very low figures. Without a study of the details one would be inclined to expect that the leakage would be large. To what small amount it can be held, however, is apparent from the guarantee made by the contractor for the rollers on the Long Lake Development of the Washington Water Power Company. On three rollers 65 ft. long and 19 ft. high the leakage is guaranteed not to exceed 2 sec.-ft. for the three. A very suitable place for rolling dams is in the Ohio River at places where dams of the "bear-trap" type are used.

Hetch-Hetchy Bill Passes The Hetch-Hetchy battle is nearing the decisive conflict. The Senate, on Dec. 6, passed the Raker bill by a vote of 43 to 25 and it now goes to the President for his signature. That every means will be employed to induce him to veto the measure is accepted by all who have followed the controversy. In view of the technical data in support of the project, and the fact that the reservoir will not destroy the beauty of the park—as every one except an intemperate enthusiast must admit—it is hoped the President will display again the sound common sense which has characterized his other acts and sign the measure. The wild statements that have been made in the press about this project show an utter disregard of the facts. The New York "Times," for example, normally sane in its comment, indulges in language that is nothing short of ludicrous. "One National Park Lost" is the title of its editorial on the Senate's action, even though John Muir has admitted that the area to be covered by the reservoir is only "about 3½ miles long and of a width varying from ¼ to ¾ mile." It compares the giving of the Hetch-Hetchy valley to San Francisco to the surrender by a city of one of its parks for "commercial purposes"—manifestly a far-fetched and unfair comparison in view of the fact that San Francisco will use the valley for a municipal water supply. Then, finally, the "Times" rants about the maintenance of a lobby in Washington and intimates that the opponents of the scheme have neglected to bring influence to bear on Congress. The latter intimation is absolutely false, as the editorial itself shows, for it says: "They [the opponents] had the press and enlightened public opinion and all men of public spirit on their side." And the press was enlisted by a well-planned and persistent campaign in which the newspapers were supplied with burning protests from Robert Underwood Johnson, John Muir, and other distinguished men. In other words, the campaign against the measure did not lack support and, in fact, was bolstered up by the most powerful agency in the country—the press. Certainly no lobby could influence Congress more than the press, pouring out daily the sentiment manufactured and sent out as "press material" by the nature-loving enthusiasts. It speaks well for the independent thinking of the Senate that it was able to withstand the assault. The President, the public knows, will approach the subject with an open mind. The weight of technical evidence, the fact that the Secretaries of Agriculture and the Interior, the Army Board of Advisory Engineers, the directors of the Reclamation Service and the Geological Survey, and the Chief Forester see no objection to the measure, and that the claims of the nature lovers are far overdrawn should convince the President that the bill merits his approval. It should be clearly understood that there is no intention on the part of this journal to attribute unworthy motives or practices to the opponents of the measure. They are enthusiasts. Their ardor has led them to exaggerate the matter, and as a result they have beclouded the issue for the newspapermen of the country, who have been guided by the "press matter" of the opponents and have not viewed the entire subject impartially.

Ideals of the Technical Schools

The thesis of Dr. Goss's presidential address before the American Society of Mechanical Engineers last week is well expressed in a sentence from the closing paragraph: "The problems of the technical school should not be left to the schoolmaster, for, roughly interpreted, they are not the problems of the school but the problems of the profession."

In proving his thesis he has set up ideals for the technical school that should not only be an inspiration to teachers but should help to convince young engineers of the possibilities in a teaching career. Of special significance were his views on the "mastery" that is demanded in the high-grade teacher and of the future of the engineering school laboratory.

The need of the technical school to-day, Dr. Goss said, was for increased quality in its instructional staff. Great leaders are few. Probably the trouble can be found in the point of view. Too often the teacher's work is looked upon as a "job," not as a career, and interest and enthusiasm are measured largely by financial returns. In saying this there is no intention of broadly indicting the engineering teachers of the country. Many of them regard their work as a profession and are devoted to it, but many more there are whose devotion is tempered by a yearning for the supposedly "green fields" of lucrative practice beyond.

Fortunately the low compensation of which teachers complain is rising. Dr. Goss realizes that the compensation heretofore has been a serious handicap to the efficiency of the technical school, and so he argues not only for higher ideals but for a substantial increase of the teacher's remuneration.

Greater financial liberality will undoubtedly have a salutary effect on the teaching staff, but the chief improvement must come by striving toward the ideals Dr. Goss has set forth. If these are partly realized the vista will open before the young man while still in college, and he will be willing, in order eventually to scale the heights of professional achievement, to prepare himself worthily, by post-graduate work in college and in the field of experience, for a university career. The ideals that Dr. Goss has set up are freedom from deadening routine and the development of the engineering-school laboratory so that it may contribute largely to the advancement of the profession. "Masterful leadership" is incompatible with the handling of petty matters, while a laboratory which is pushing forward the bounds of human knowledge is an inspiration to every man in the institution, student and professor alike. One who has talked to the students of schools which have contributed substantially to the fund of professional knowledge will appreciate that in none of their institution's achievements do they take a greater pride. Dr. Goss's words in this connection deserve special note.

"A laboratory," he says, "which has been evolved through the activities and desire of a master is not only priceless for the school that possesses it, but necessary as a source of information of the highest value to the field of practice which it is designed to serve. . . . Assuming that the country has need of research laboratories for purposes quite apart from the work of instruction, it can, I think, be shown that the technical schools constitute the most promising agency in our national economy upon which to place the responsibilities incident to their creation and maintenance."

A reading of Dr. Goss's paper, abstracted on page 658, will convince anyone of the influence the technical school has had upon professional ideals. If further evidence is wanted one need only point to the fact that the presidents of three of the great national societies are teachers, Dr. Goss of the Mechanical Engineers, Professor Swain of the Civil Engineers, Professor Talbot of the Society for Testing Materials, while Professor Kemp held the presidency of the Institute of Mining Engineers last year. What clearer evidences of the triumph of the technical schools and their place in the profession? It is an inspiration for attaining the ideals that Dr. Goss has briefly sketched.

Limiting the Height and Area of Buildings

Probably no city in the country has appreciated more than New York the need for establishing some limit for the height and area of buildings, and none has been more remiss in the face of conditions rapidly reaching the unbearable. Only those who have tried to hurry through the narrow streets of lower New York during the rush hours or have been obliged on a bright day to toil for seven or eight hours under electric light even alongside of a window can appreciate the seriousness of conditions. An aggregation of narrow canyons has been created. As many as twenty years ago the warning was sounded, but not until last week did there appear any hope for sane action; real solution became impossible years ago, for much of the damage is irreparable. That the subject is of widespread interest is apparent from the attention given to it, not only in New York, but elsewhere. An amendment has been offered to Chicago's 200-ft.-limit ordinance which would permit the erection of buildings over 200 ft. high, provided there is a setback of 5 ft. for every 100 ft. of additional height. In Montreal a proposal has been made to limit the height to the width of street, while in Seattle a 200-ft. limit has been proposed.

The Board of Estimate and Apportionment of New York City appointed from its own membership some time ago a committee to consider the subject. That committee appointed a Heights of Buildings Commission, composed of nineteen members—architects, building owners, real-estate operators, contractors, students of housing problems and one engineer—the latter being Mr. Nelson P. Lewis, chief engineer of the Board of Estimate. The commission has now rendered a unanimous report, which is abstracted on page 659.

There are two main features of the document—first, a set of rules recommended for immediate adoption looking to the limitations of heights and areas, and, second, a recommendation that the city's charter be amended so that building districts may be created, in which special restrictions will be imposed in accordance with the character of the neighborhood. In reading the abstract referred to it must be constantly borne in mind that the suggested rules were framed chiefly with the financial district in mind. They are manifestly inadequate for the city as a whole and are recommended for universal application only until such time as the law will require suitable districting and more stringent local regulations.

The height limit is set as twice the width of the street, but such limit shall in no case be less than 100 ft. nor more than 300. After reaching this limit the buildings may be carried higher by setting the street walls back 1 ft. for each 4 ft. of increased height.

There are no area limitations for the first story; it may occupy the entire lot. Above the first story 10 per cent of the rear of every interior lot must be left vacant. In addition there must be a further loss of area covered by the building equal to 1 per cent of the lot area for each story except the first, but the loss of area occasioned by setbacks of the front walls is included in this 1 per cent required. The area requirements are supplemented by prescribing minimum dimensions proportionate to height for main courts other than the 10 per cent rear courts. The least dimension of such courts must be not less than 6 ft. and not less than the number of feet equal to one and one-quarter times the number of stories above the first. These court requirements and the 1 per cent loss of area clause do not apply to lots of special shapes and sizes and for which adequate light and air can be secured from the street.

As an exception to all the above-mentioned height and court regulations a tower may be erected to any height, provided it does not cover more than 25 per cent of the lot and provided every part of it is kept at least 20 ft. from the lot and street lines. In buildings facing parks or the waterfront the tower may be placed at the building line.

From a reading of these details it would seem that the considerations of light and air were chiefly in the minds of the commission, but a study of the effects of the clauses will indicate that the question of congestion has been provided for. The commission estimates that for plots of normal size the setback and area regulations will limit the height when the building area has been reduced to about 60 per cent of the area of the plot. For a building on an interior plot on a 60-ft. street the economic height limit will be from 14 to 17 stories, while on a corner plot on a 100-ft. street the economic height limit will probably be 16 to 20 stories. As to towers, the commission feels that the restriction will prevent their interference with light and air and that they will not prove attractive investments. Their construction, therefore, will be discouraged except for purely advertising reasons, which undoubtedly have prompted the erection of some of the very tall buildings in New York and other cities.

In order to estimate the relation of these restrictions to those adopted elsewhere in this country it may be noted that Boston has placed the maximum at two and a half times the street width, but not over 125 ft. Denver has placed it at 12 stories; Los Angeles at 150 ft., Baltimore at 175 ft., Chicago, Cleveland and Newark at 200 ft., while in Jersey City and New Orleans the limit is two and a half times the width of street.

It is apparent, therefore, that the recommendations of the commission are a compromise. Certainly a 120-ft. building on a 60-ft. street or a 200-ft. building on a 100-ft. street does interfere very considerably with light and air. Moreover, a district built up under these regulations will be even more congested than is the lower section of Manhattan to-day. Unquestionably the regulations would have been more drastic had it not been necessary to protect the property interests in the congested portion of the city. Very likely, too, more severe restrictions would have met a crushing opposition from owners and real-estate operators. A limiting first height equal to width of the street, or, at most, to one and a half times that width, with a maximum limit of 150 ft., would have been much better from every consideration—light, air, congestion and fire protection. Nevertheless, it is apparent that the real-estate interests must have made what they regard as very large concessions in agreeing to the report.

That the commission, as a whole, believes firmly in a much lower cornice line is evident from the part of the report recommending the districting of the city. The necessity for more stringent regulations is pointed out, while the segregation of industries is insisted upon. Anyone who has suffered heavy losses in a residence property because of the advent of a factory will sympathize with this part of the commission's report. An amendment permitting districting should be adopted at the coming session of the State Legislature.

As to the specific recommendations, if Father Knickerbocker is wise he will adopt them post haste. While not thoroughly satisfactory to the advocates of the city beautiful or the real-estate owners, they represent a reasonable compromise, considering conditions now existing in lower New York.

Naturally the thoroughness of the investigation will attract widespread attention throughout the country. In considering it with a view to applying its recommendations to other cities, the Engineering Record would again emphasize that it must be clearly understood that the suggested rules were framed with the needs of lower New York in mind. They are far too liberal for even the business district of any other American city. A first limit of 150 ft. might well be the maximum.

Public Works—for Service or the Builder's Profit ?

The reasons for desiring the building of public works are of two kinds—first, a desire for the advantages that will grow out of the completed works, and, second, a desire for the advantages and profits of construction.

By common consent, only reasons of the first kind are officially and publicly urged in support of any given project, and it is well that this is so; but no one actively engaged in public work can be blind to the fact that reasons of the second kind are often—in fact much too often—more potent in securing actual results.

Both kinds of reasons may be legitimate. The contractor expects a profit and has the full right to look for work that will yield it, and within reason to do what he can to advertise and promote the kind of works that he can build. The engineer and the architect, also, have a similar interest in the commissions that may come to them; and all those who may be employed, either by a public authority or by engineers and architects, or by contractors, have their interest in seeing matters move forward. Those who have land to sell, those who furnish all of the thousand kinds of supplies used in construction, and even the bankers who make a commission by the sale of bonds—all have an interest in seeing the expenditure of public moneys go forward.

Financial rewards are by no means the only ones that need be considered. The experience, the honor, the prestige, and the political power that come from carrying out works, especially when they meet with popular approval, are often potent in securing the co-operation of those who would not be influenced by financial considerations.

Up to a certain point all this is well and tends to progress and civilization. The danger lies in the chance that when the forces of construction that are necessary for progress are organized and developed they will come to have a dominating influence in the transaction of public business, and will secure the construction of public works that could not be justified from the standpoint of the results to be obtained from the finished works, and will do this for the sole, or principal, purpose of yielding the profits that grow out of construction. When a government reaches this stage it ceases to be a government for the people and becomes a kind of piracy in which the taxpayers are plundered.

There is need at this time as never before, in view of the slackness of private enterprise, of engineers and other citizens with breadth of view to rightly appraise the need for public works and with the honesty and courage to speak fearlessly concerning them.

Future of the Motorbus

The operation of motorbuses in city streets has almost as many points of interest to the municipal engineers as to the operators of electric railways. Both the bus and the street railway have an influence on traffic congestion, population congestion and safety on the streets. In addition, the maintenance of pavements is a factor in bus operation. For these reasons some features of a report recently rendered to the Mayor of New York by the bureau of franchises deserve the attention of the municipal engineers.

The bureau sent abroad in July of this year one of its engineers, Mr. John A. McCollum, in order to gather first-hand information on the subject. His findings are based on observations in London and Paris, and are of moment in New York at present because four franchises for bus operation are now pending.

Probably the most important and immediately applicable finding is the very sensible one that franchises should not be granted unless the new service will fill a place in the transportation field not already supplied, or, if it is to duplicate on existing service, it must be more complete, efficient, economical or comfortable. It is very properly pointed out that efficient street-railway service cannot be furnished if the property is not prosperous.

But even if it were accepted that competition might be permissible, the case of the motorbus as a "more complete, efficient, economical, comfortable or otherwise better facility" would, judging by Mr. McCollum's findings, be difficult to prove. The bus, because of its limited capacity, is unable to provide rush-hour facilities as effectively as the street car of larger capacity—assuming that the weight and capacity of motor vehicles are rigidly limited, as in London. Any attempt to provide an equally good service will result in increasing the number of moving vehicles over what would be required with street cars, with consequent increase in traffic congestion. One of the most trying problems in all large American cities to-day is the increasing congestion of the streets.

Additional objections to the motorbus are the greater vibration, with consequent discomfort, and the decidedly adverse accident record. The latter point is strongly emphasized in the report; and while accidents can undoubtedly be reduced, it is only at the price of speed restrictions and other traffic regulations which will work still more adversely to the motorbus as a traffic carrier.

On the other hand, it must be admitted that there are arguments in favor of the motorbus, though hardly strong enough to overbalance the objections. It can easily adapt its movements to street traffic, and two buses can run abreast at times, thus clearing dense-traffic intersections much more rapidly than street-railway cars. The breakdown of one vehicle does not derange the entire system, while point-to-point speeds are somewhat higher.

Taking the report as a whole, one would be inclined to believe that the outlook for the motorbus is not favorable. Certainly when franchises are granted it will be with restrictions that will safeguard the form of transportation which has been shown to be more generally satisfactory—the street-railway car. The report definitely states, nevertheless, that there are routes in New York upon which the motorbus can supply a needed service in local areas. With proper restrictions the public can benefit by the satisfactory features, while, at the same time, the disadvantages are reduced to a minimum.

Progress on Pennsylvania's State Highways

Work Accomplished by Department Organized under Sproul Act of 1911 and Notes on Organization, Construction and Maintenance

THE recent defeat by the voters of Pennsylvania of a constitutional amendment authorizing a $50,000,000 bond issue for highway improvements justifies a statement of the progress made on the road system of that State, together with an account of some of the methods in use by the highway department.

Prior to June 1, 1911, the date on which the Sproul Act became operative, funds for highway improvement by State aid were appropriated by the Pennsylvania Legislature from time to time from general funds. The Sproul Act, under which improvements have since been made, provided for the establishment of a State highway department, to be administered by a commissioner, two deputy commissioners, a chief engineer, an auditor, a chief draftsman, fifteen assistant engineers, superintendents of highways and a staff of assistants and employees. The act designated 296 routes, to be known as State highways, connecting county seats and principal towns and cities and included a State-aid provision whereby townships and counties might petition the State for road improvement, one-half the cost of which was to be borne by the State. By an act approved July 22, 1913, seventy-eight more State routes were added. The present total mileage contemplated is about 9,100. The bill further provided that the expenses of the department, when properly certified by the commissioner and audited and allowed by the auditor-general, should be paid out of the funds specifically appropriated for this purpose.

Table 1—State Highways Completed or Under Contract Oct. 30, 1913

(Work begun June 1, 1911)

	Water-bound macadam	Brick	Asphaltic macadam by penetration	Asphaltic concrete	Concrete
Length in miles	86.23	8.94	48.79	41.17	2.71
Total cost	$717,404.95	$232,477.12	$759,820 86	$885,698.42	$50,718.62
Cost per mile	8,319.60	26,004.15	15,573.08	21,513.20	18,715.30

Construction

Since the Sproul Act came into operation, 280 miles of surfaced highway have been laid by the construction department at a total cost of $4,261,138.07. Of this amount $2,646,119.97 was expended in the building of 187.84 miles of State highways. In addition, there had been contracted for or completed under the law in force prior to the formation of the present department 118.01 miles of State-aid highways, at a cost of $1,656, 118.66; 22.7 miles of the National Road, at a cost of $153,027.54; and 0.25 mile of experimental concrete highway at a cost of $4,872.57. The classes of roads laid prior to Oct. 30, with their respective mileage and the cost per mile, are given in Table 1.

The present practice is to build all roads 16 ft. wide; but this does not apply to all the roads in Table 1. Some of the brick sections were a good deal wider than 16 ft.; this accounts for the unusual cost per mile. The figures for the water-bound macadam include some maintenance contracts, all in cases where the foundations were partially rebuilt. Table 2 gives corresponding data for State-aid highways during the same period.

A study of the two tables shows that of the total amount of work done by the construction department the class calling for the greatest expense has been asphaltic macadam by penetration.

On both State highways and State-aid roads $1,536,943.21 has been expended to build 95.21 miles of this type. Of asphaltic-concrete roads 50.07 miles have been built at an expense of $1,090,466.18; 102.3 miles of water-bound macadam have cost $864,424.32; 29.3 miles of brick, $711,181.74; and 3.16 miles of concrete, $58,122.62. Water-bound macadam is gradually being eliminated.

A telford foundation is used for all water-bound macadam roads, employing stones to a depth of 8 in., laid by hand with the broadest edges down and lengthwise across the axis of the road in the form of a rubble pavement. These stones are thoroughly bound into place and then wedged with smaller stones. Where concrete foundation is used it is at least 5 in. thick. For the ordinary surfacing the stone required is between the sizes 1½ in. and 3 in. rolled to a thickness of at least 4 in.

An asphaltic concrete pavement is laid upon a telford or concrete foundation. The asphaltic concrete mixture is composed of from 53 to 62 per cent of crushed stone, from 30 to 37 per cent of sand and from 8 to 10 per cent of asphaltic cement. The stone used is run-of-crusher passing a 1½-in. ring and containing not more than 5 per cent of dust. The asphaltic cement is heated to from 275 to 375 deg. Fahr., and is spread at a temperature of not less than 225 deg. with hot iron rakes. Ten-ton rollers are used for all rolling.

As soon as possible after rolling, while the surface is still fresh and clean—if possible, while warm—a seal coat of bituminous cement of proper consistency to be flexible when cold is spread over the surface at a temperature of from 200 to 350 deg. Fahr.

When brick pavement is used the brick is subjected to the rattler test as recommended by the National Paving Brick Manufacturers' Association, as well as to absorption and crushing-strength tests. Expansion joints ¾-in. wide are required along each curb.

Maintenance under the present form of supervision is performed entirely within the department. This system dates only from April 1 of this year and detailed data on the work will not be available until the January departmental report is issued. It may be said, however, that about $2,000,000 has been spent in organizing the department, purchasing machinery and actual maintenance.

Table 2—State-Aid Highways Completed or Under Contract Oct. 30, 1913

(New State-Aid Fund)

	Water-bound macadam on telford	Brick	Asphaltic macadam by penetration	Asphaltic concrete	Concrete
Length in miles	16.07	20.36	46.42	8 90	.45
Total cost	$147,019.37	$478,704.62	$777,122.35	$204,767.76	$7,404.00
Cost per mile	9,150.00	23,020.85	16,741.00	23,007.61	16,453.33

All maintenance work is under the supervision of a maintenance engineer, to whom 50 superintendents report. Each maintenance district is divided into sections of approximately 50 miles, each with a general foreman in charge.

A large working plant has been assembled—40 10-ton rollers, 20 crushers, 122 wagons, 62 road machines, 1500 road drags, 23 scarifiers, 44 sprinklers, 11 motor trucks, and 2 motor oil distributors. The success which the department feels it has met makes probable the rapid expansion of this plant.

Growth of Motor-Traffic Receipts

A large part of the funds devoted to maintenance work is derived from the registration of motor vehicles, special registration of dealers, and from licenses for drivers. The amount thus derived is placed in a separate fund to be used for no other purpose. In 1912 the State treasury collected $1,700,000 from these fees. Owing to a delay caused by a ruling of the auditor-general, who contends that the wording of the act of May 23, 1913, does not constitute a specific appropriation, the money at present in the State treasury from this source has been tied up and the question of its use is now in the hands of the court. In March, 1913, the $1,800,000 then on hand was specifically appropriated to the highway department.

The fees are levied as follows: Motorcycle, $3; motor vehicles with pneumatic tires, up to 20 hp, $5; up to 35 hp, $10; 50 hp, $15; over 50 hp, $20; motor vehicles with solid tires, up to 4000 lb. gross load, $5; up to 5000 lb., $10; up to 10,000 lb., $15; 15,000 lb., $20; and between 15,000 and 24,000 lb., $25. Vehicles trailing after motor vehicles are taxed at the rate of $3 for each weighing less than 10,000 lb. and $5 for each of less than 24,000 lb. Traction engines are taxed $10 up to 20,000 lb. and $20 between 20,000 and 26,000 lb.

Table 3—Growth of Motor Receipts, Jan. 1, 1910, to June 1, 1913

Year	Registered auto owners'	Registered dealers	Registered owners' motorcycles	Registered drivers	Receipts
1910	33,346	2,908	3,834	16,271	$321,985.12
1911	43,282	4,034	4,826	17,051	429,523.50
1912	58,221	3,555	7,298	21,235	597,723.19
1913	64,405	2,975	7,861	23,716	682,689.37
Total receipts, Jan. 1, 1910, to June 1, 1913					$2,031,921.18

The great increase in the number of motor-driven vehicles is shown by Table 3. The figures for 1913 extend to June 1 only.

Experimental Surfaces

About two miles of the old Baltimore road in the southern part of the State was resurfaced in April, 1913. The work included scarifying the water-bound macadam surface and applying two 1½-in. layers of bituminous surfacing. The old road surface was brought to shape for 12 cents per square yard and the new bituminous surface was applied at about 55 cents per square yard, making the total cost of improving this road 67 cents per square yard. This work was in the nature of an experiment and will be watched carefully by the maintenance department during the next few years.

Further experimenting was carried on in Erie County, where two ¾-mile stretches of water-bound macadam roads were re-surfaced. In each case 4 in. of 2-in. stone were used under a coat of Class B asphalt, spread in the proportion of 2½ gal. per square yard. On one stretch the penetration method was used; on the other the materials were hand-mixed. The penetration-mixed surface cost 76 cents per square yard to apply; the hand-mixed surface cost $1.08. As the roads were only laid in June of this year, it will be some time before comparison of the two methods may be made safely.

A system whereby each individual of the department reports his activities at frequent intervals is in operation. Each laborer has a time card; each day the foreman in charge must fill out a blank reporting the activities of each laborer and the time spent by him. From these reports the superintendent is required to compile a weekly report giving in detail the work accomplished and setting forth the work contemplated for the ensuing week. another feature of this report is a weekly traffic census, which is designed to indicate the most suitable type of road. At intervals of a month each superintendent and general foreman must submit a record of the machinery and tools on hand at the end of that time. All these records are carefully cross-checked in the maintenance engineer's office and are presented to the auditor for filing.

When a maintenance section is completed a detailed cost statement is prepared in the office. The blank form used in making out this cost system is shown in the first column, together with a similar form used for motor trucks.

A comprehensive instruction booklet is issued to superintendents and foremen in both the maintenance and construction departments. The text refers to important points of roadmaking, such as grading, placing foundations and surface, and drainage, and in addition gives full instructions as to how to fill out the numerous reports.

The officers now serving in the State highway department are: Edward M. Bigelow, State highway commissioner; Joseph W. Hunter, first deputy commissioner; E. A. Jones, second deputy commissioner; Samuel D. Foster, chief engineer; William R. Main, auditor; Willis Whited, bridge engineer; P. M. Tebbs, assistant engineer; George H. Biles, maintenance engineer, and Leon F. Neefe, chief clerk. W. R. D. Hall is statistician.

PENNSYLVANIA STATE HIGHWAY DEPARTMENT
MAINTENANCE DEPARTMENT

...............No. ....Unit Cost per yd...........
Route ....Location ....
Appl.
Dist. ....Date ....191....

	ITEM	QUANTITY	COST
STRIPPING	Supervision		
	Labor		
	Supplies		
QUARRYING	Supervision		
	Labor		
	Supplies		
	Transportation		
CRUSHING	Supervision		
	Labor		
	Fuel		
	Supplies		
	Maintenance		
ROLLING	Supervision		
	Operation		
	Fuel		
	Supplies		
	Maintenance		

Total Cost
Output
Output
Output
Output Total Cu. yds.
Surface Rolled Sq. yds.
Signed
Title

PENNSYLVANIA STATE HIGHWAY DEPARTMENT
MOTOR TRUCK DAILY RECORD

Route Truck No...........
Appl....................
Date...................191....

	Check of Trips	Ton Miles
Load....tons Round Trip ......Miles		
Load....tons Round Trip ......Miles		

Total Ton Miles

Item	Quantity	Cost
Driver		
Storage		
Repairs		
Equipment		
Gasoline		
Oil, Heavy		
Oil, Light		
Grease, Cup		
Grease, Graphite		
Supplies		
Tires		
Total		

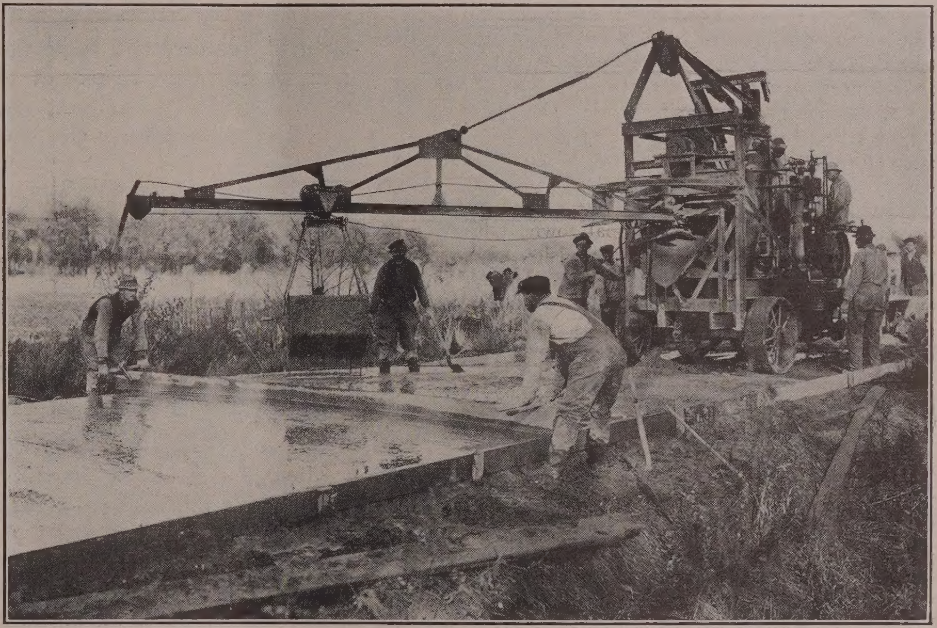
Unit cost per ton mile
Driver

BLANK FORMS FOR MAINTENANCE AND MOTOR-TRUCK COST REPORTS

Concrete Road in Cook County, Illinois

An experimental concrete road 4240 ft. long was laid last October on the Church Street Road west of the city limits of Evanston, Ill., in Cook County. Connection with Ridge Road is made at a point 2¼ miles west of Evanston, providing an outlet for automobile traffic to the northwest and tapping a rich truck garden area. The road was financed jointly by the Evanston Commercial Club, Cook County and Niles Township.

With the exception of a slight incline for a few hundred feet at the Ridge Road end the profile is level, the roadway having been graded years ago, 4 to 6 ft. above the marshy surface, having well defined ditches on each side. The soil is typical sticky black Illinois gumbo, but, with the exception of 1100 ft., the concrete was placed on an old macadam road.



A Sawing and Dragging Motion Is Used in Striking Off Pavement with Template

Single course slabs, 25 ft. between expansion joints, 16 ft. wide, 6 in. thick at the edges and 8 in. thick at the center were placed on a flat subgrade compacted by a 10-ton roller. Little grading was done until just prior to the laying of the concrete, this part of the work being carefully inspected to see that the full depth of concrete was obtained. Practically speaking, the concrete was placed on the old roadbed, which consisted largely of 3 to 6 in. of crushed stone and blast furnace slag on 18 to 24 in. of loose soil underlaid with a thin stratum of sand and gravel.

Mention should be made of the 1100-ft. stretch of poor subgrade which consisted of a black soil only in which the wheels of the mixer sank to the hubs whenever the machine ran off the planks. It is believed reinforcing bars should have been used to prevent cracking due to settlement across this section of low land, although the deep side ditches will minimize frost and drainage difficulties.

Aggregate consisted of washed bank gravel and sand, which tests showed to be first class material. It was obtained by the contractors, M. Foley & Company, from the Richardson Sand Company pits near Carpenterville, Ill. A 1:2:3 concrete, using Universal Portland cement, was mixed in one-bag batches in an Austin cube mixer. Quantities were checked with a cubic foot box at least one each day in the presence of the workmen so as to guide them in loading the wheelbarrows. Baker protection plates, separated by Barrett felt, ¼ in. thick and 8 in. wide, were held up with the Baker installing device. The felt had to be trimmed to conform to the finished crown. Water was obtained from wells dug along the roadside and pumped to the mixer by a Fairbanks-Morse gas-engine outfit.

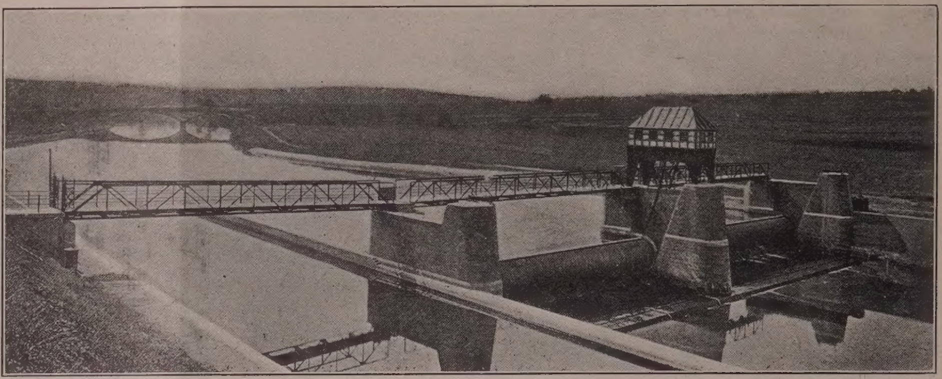
Striking Off Surface

Two men struck off the pavement with a template made of a 2 x 8-in. plank bound with a steel angle. Grasping the cross-bar handle at each end they gave it a simultaneous sawing and dragging motion. This produced an almost finished surface. Little was left to be done by the finisher who followed with a wooden hand float, working from a bridge.

Most of the finished pavement was covered with earth or sand and sprinkled at least once a day. More frequent sprinkling was considered unnecessary on account of rains during the night and the low temperature. Usually concrete laid the previous day was hard the next morning, indicating no delay in setting. On account of lowered temperature and rainy weather concrete deposited after Oct. 22 was not covered with earth or sprinkled.

Earth from the side ditches, which were put into good condition, was used in conjunction with gravel hauled from a distance for the 4-ft. earth berms. This material was soft and will require much compacting, particularly as the edges of the concrete were not pared off but left vertical as placed against the 2 x 6-in. side forms.

Work was initiated Oct. 1 and the concrete slabs were completed on Oct. 24. Rates of progress varied from 90 to 575 lin. ft. per day, averaging about 230 ft. The average crew was one finisher, one engineer, one fireman, three spreaders, three graders, one charging skip, one supplying cement, six wheeling sand, one mixing, two setting up joints, two form setters, one superintendent and one foreman, part time only.



Rolling Dams in the Neckar River near Poppenweiler, Germany

Design of Rolling Dams

General Features and Important Details; Methods of Calculation and Data on Structures Built and under Construction

By A. G. HILLBERG, C.E.
Editorial Staff of the Engineering Record

CONSTRUCTION of a rolling dam on the Boise irrigation project, as described in the Engineering Record of Aug. 2, 1913, page 125, and adoption of rollers for the Long Lake development of the Washington Water Power Company and for the Grand River dam of the U. S. Reclamation Service bring this type of structure into new prominence. While the rolling dam is not entirely new, the first having been built in 1902, only little has been published on the subject in English, save the paper read by Prof. K. E. Hilgard, of Zurich, Switzerland, before the International Engineering Congress in St. Louis in 1904. For that reason an effort has been made to treat the matter comprehensively in this article. Such treatment is needed because an entirely new type, with an enlarged sheath or skin plate, has been developed since Professor Hilgard's paper was presented.

General Features and Advantages

The main feature of a rolling dam is a watertight steel cylinder, which rolls on inclined rails supported in lateral recesses in masonry piers or abutments. In its lowered position it serves as a dam and when opened is suspended out of reach of floating matter. It is manipulated by a chain or wire rope wound around one end. The uniform advance of both ends in raising or lowering is insured by gearing, each end carrying a toothed rim engaging a rack fixed on the masonry.

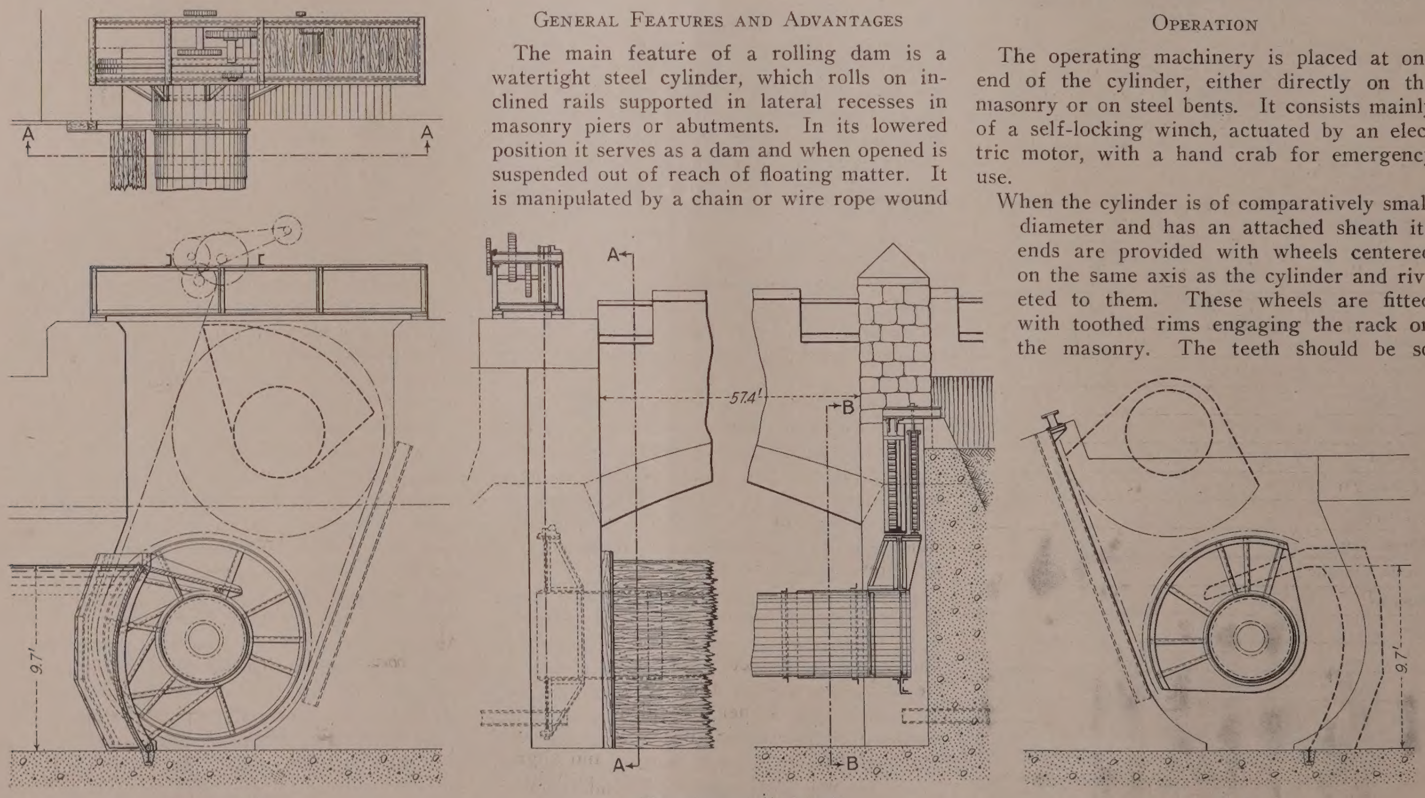
Rolling dams have many advantages, of which the following are the most important: Possibility of building discharge openings of large spans and depths; absence of obstruction to débris and ice when the dam is open; rapidity in operation; low operating resistance; great durability and stability; small leakage; certainty in operation, and low attendance and maintenance costs.

At first the rollers were plain cylinders, which necessitated their filling with water, or otherwise counterweighting, to prevent them from floating. In later designs the cylinder has a comparatively small diameter. On its upstream side there is a waterproofing sheath or skin made of steel or wood, shaped so as to prevent any resulting uplift on the roller.

Operation

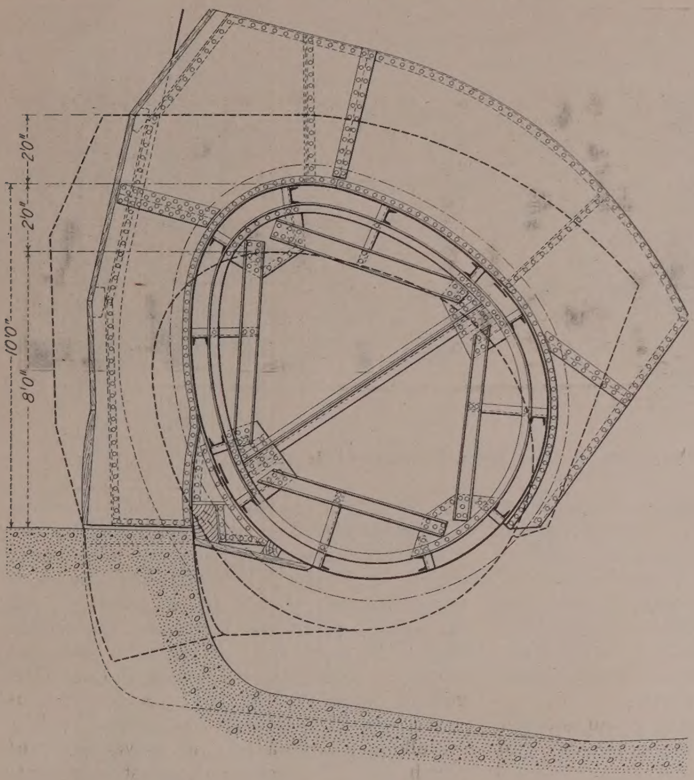
The operating machinery is placed at one end of the cylinder, either directly on the masonry or on steel bents. It consists mainly of a self-locking winch, actuated by an electric motor, with a hand crab for emergency use.

When the cylinder is of comparatively small diameter and has an attached sheath its ends are provided with wheels centered on the same axis as the cylinder and riveted to them. These wheels are fitted with toothed rims engaging the rack on the masonry. The teeth should be so

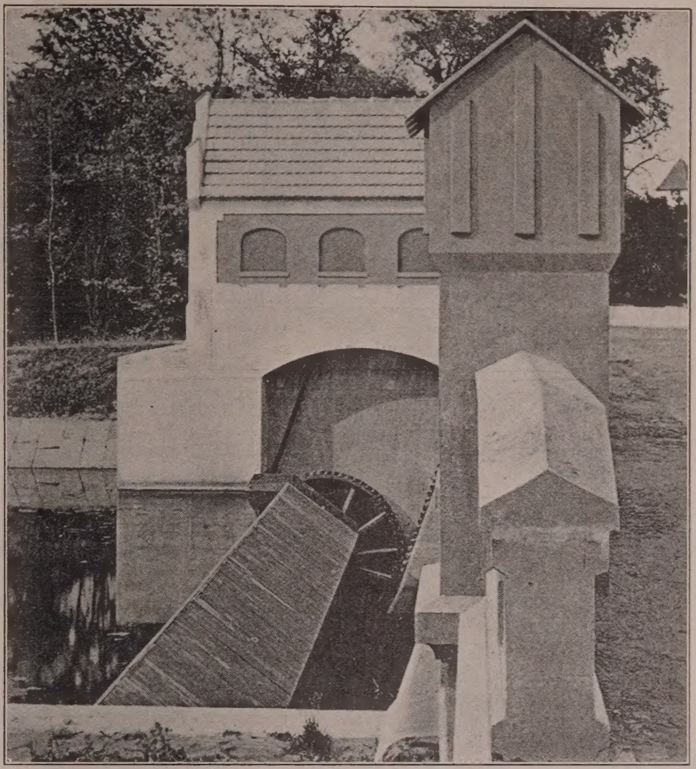


Section A-A Elevations of Abutments Section B-B

Details of Rolling-Dam Installation at Neugattersleben on the Bode River in Germany



Egg-Shaped Roller for Rapidity of Action

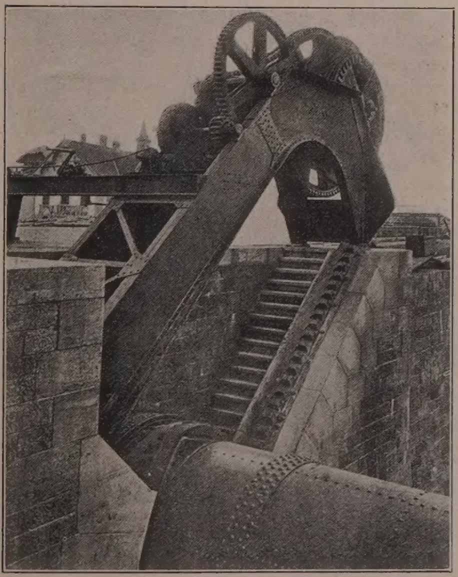


Rolling Dam with Wooden Sheath at Neugattersleben

designed that any foreign matter getting into them will drop through or can easily be removed. They are, therefore, of a different design than machinery gears, as will be seen from the illustration.

Either wire ropes or sprocket chains are used at both ends of the roller. The chain or rope at one end is connected with the operating machinery and serves to manipulate the roller. The chain at the other end, however, is a safety precaution only. The upper end is anchored to the masonry and the lower is bolted to the roller in such a way that when the roller is raised the chain is wound around it. Should an accident befall the racks or deform the roller, or foreign matter fill the teeth so that the toothed rim does not engage the rack, the roller will be suspended on the chain and will operate as if everything were normal.

To care for the slack which has already passed the sprocket of the driving mechanism the chain is fitted at intervals with projecting pins, as shown in the illustration.



Details of Rolling Dam at Schweinfurt

For a very deep opening it will sometimes be advantageous to place two rollers, one on top of the other, as illustrated in one of the drawings.

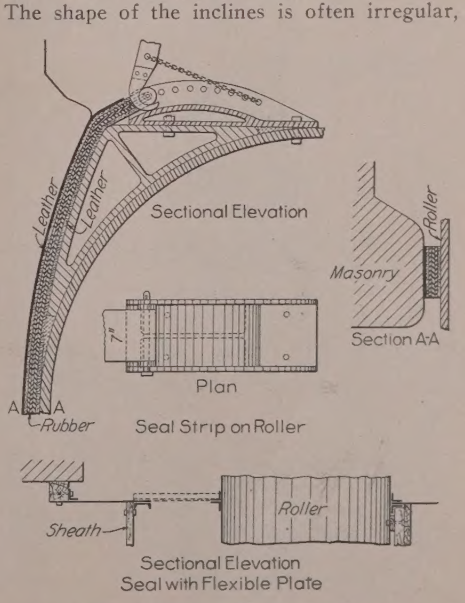
When it is desired to skim the pond without raising the roller its full height, as, for instance, at an ice sluice, the roller can be designed so that it can be depressed a certain amount. One of the figures shows such an installation, where the roller is of an egg-shaped cross-section so as to give a larger depression for a small movement than if it had been cylindrical.

METHODS OF SECURING TIGHT SEALS

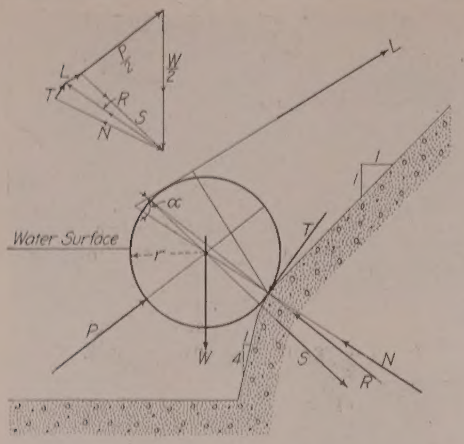
To insure a nearly perfect seal along the bottom is not difficult. It is accomplished by bolting to the roller a strip of wood, which bears on the masonry dam below. As nearly the entire weight of the roller in the closed position comes on this strip, a good tightening pressure is insured. No débris or detritus can collect between the sill and the tightening strip and so prevent an absolute seal, because the lower the roller sinks the more violent is the rush of water underneath, until, at the moment of closing, the water is rushing through under full-pressure head. To insure a properly aligned surface on top of the permanent dam it can be provided with a cast-iron bull beam, machine finished on top. Should the rolling dam consist of a plain cylinder, it will be found advantageous to place the wooden tightening strip on the sill instead of on the roller. This, on the other hand, is inconvenient in case of repairs, while a tightening strip on the roller is always accessible when the dam is open.

Several methods are used for securing a seal at the sides. One of the first schemes was to build the recesses of such shape that the seal was made on the surface of the cylinder, which was then provided with elastic strips, as shown in the illustration. But as it is very difficult to build the roller and align it so that it will at the same time take bearing along the sill and in the recesses, this method has been abandoned in favor of schemes permitting the roller to take bearing along the sill and to seal the sides even if the wooden strips on sill and sides gradually wear off. One way to secure a seal on the sides is to provide the roller with large shields normal to its axis. These shields are made of thin steel plates elastic enough to permit the water pressure to deflect them. Around their edges are wooden strips, which press against finished surfaces on the masonry piers. This type gives such good results that manufacturers guarantee the leakage to be from 0.005 to 0.01 sec.-ft. or less per linear foot of seal.

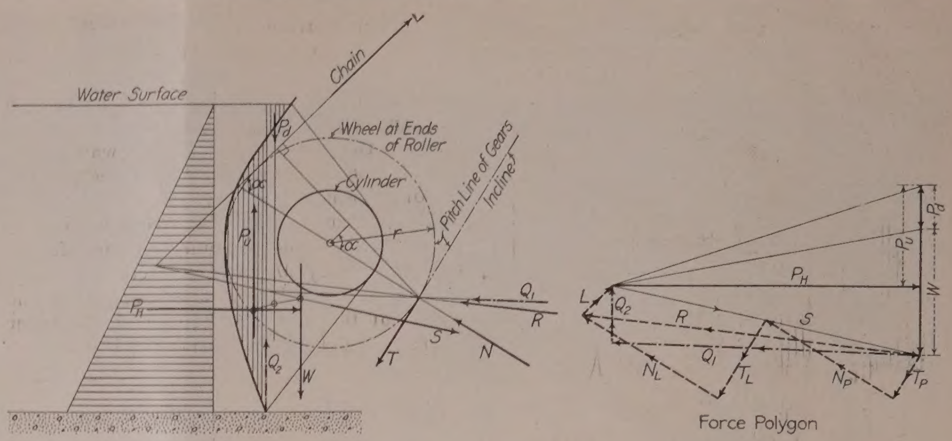
At both ends of the roller are roomy recesses in the sides of the masonry piers. The shape of these recesses depends on the shape of the sheath of the roller, since the sheath when the roller is being opened should not encroach upon silt, débris or ice on its upstream side. These recesses contain the racks, ropes or chains and safety appurtenances necessary for the operation of the roller. Furthermore, to make the roller easily accessible stairways are located along the inside of the racks. The shape of the inclines is often irregular,



Water Seals at End of Roller



Pressures on Plain Cylinder



Pressures on Rolling Dam Provided with Sheath

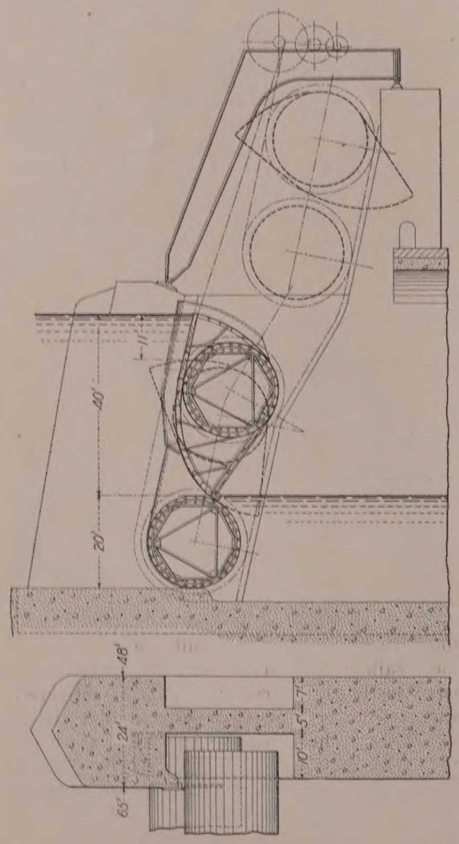
and more so if the roller is a plain cylinder, with the side tightening strips fastened to the cylinder. All parts susceptible to injury are within these recesses and therefore accessible.

During heavy winters the seepage around the sides of the roller freezes and might cause trouble if precautions are not taken. It has been found, however, that even in a climate as severe as that of Sweden ice can be prevented from adhering to the metal surfaces of the roller, and binding it, by placing a coal stove in the part of the roller which projects into the recess. As the tightening strips are of wood they are not subject to serious adhesion. The fact that ice troubles can be so easily avoided makes the rolling dam adaptable to cold climates, where other movable dams must be thawed out.

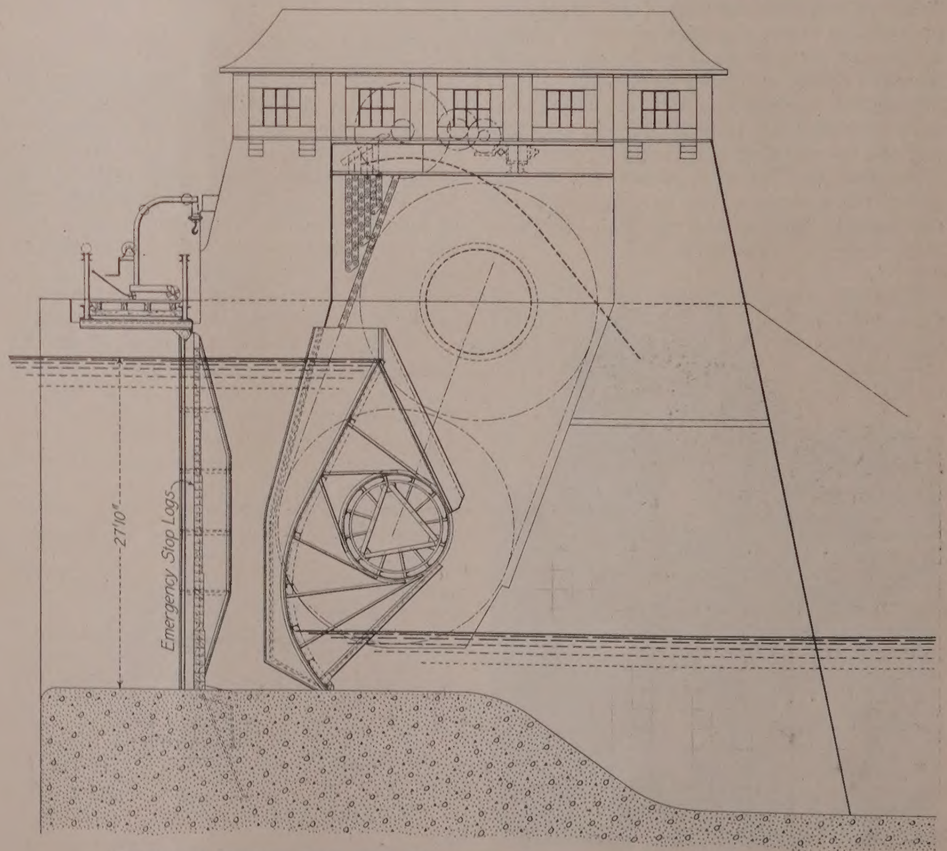
In figuring a rolling dam one of the fundamentals is that in the closed position the resultant force does not tend to roll it up the incline. The second dam at Schweinfurt (see table) has, therefore, a curved incline, the upper part of which has a slope of 1 on 1 and the lower part a slope of 4 on 1. These slopes are connected by a curve of 10-ft. radius. In the figure marked "Pressures on Plain Cylinder" the action of the forces and their resultant are shown. *W* is the weight of the roller; *P*, the resultant water pressure; *S*, the resultant of *W*/2 and *P*/2; *L*, the strain on the chain, and *R*, the reaction due to *W*/2, *P*/2 and *L*. This reaction is resolved into two components—*T* and *N*—tangent and normal to the incline.

The forces due to water pressure on a roller have been much discussed by engineers. Many still adhere to the so-called "normal theory," but in a paper published in the "Oesterreichische Wochenschrift für den Oeffentlichen Baudienst," and later printed in the Engineering Record of March 8, 1913, page 279, Prof. E. Jacoby, of Riga, Russia, proves that this theory is wrong and recommends the application of two forces. One of these is equal to the horizontal water pressure on a perfectly plane vertical wall of the same depth as the submerged part of the roller, and the other a vertical force equal to the displacement. These forces combined with the dead load of the roller give a resultant force, which must be divided between the supports of the roller. As the roller when the chain is without stress is supported along its bottom contact and at the side contacts on the incline, it is easy to determine the amount of these reactions Q_1 and Q_2, of which Q_2 will be equally distributed along the sill and Q_1 equally between the ends of the roller (see drawing entitled "Pressures on Rolling Dam Provided with Sheath").

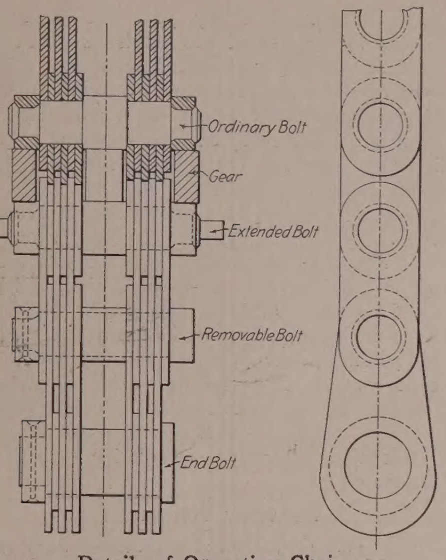
As soon as tension is put on the chain—i.e., when the roller is being opened—the load on the sill is relieved and the force due to dead load and water pressure will be divided into one component along the direction of the chain and another through the points of contact of the roller and the inclines. The components, normal and parallel to the inclines, can now be determined. It is to be noted that the tension in the chain is a force applied to one end of the roller only, thus making the component forces larger at that end. The difference is clearly shown in the force polygon. In the diagram P_h is horizontal, P_u, upward, and P_d, downward water pressures; *W*, dead load of the roller; *S*, resultant of the forces *P* and *W*; Q_1 and Q_2, reactions due to *S*; *L*,



Dam with Double Rollers



Details of Rolling Dam with Sheath at Kibling on the Saalach River



Details of Operating Chain

stress in chain; T_l and N_l, tangential and normal forces acting at the chain end of the roller, and T_p and N_p acting at the other end.

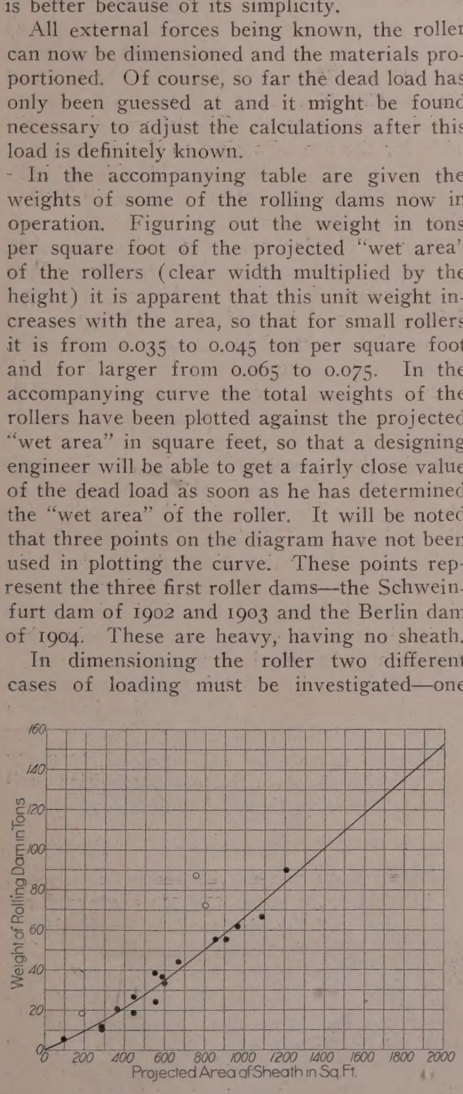
It will be noted that in both of the diagrams referred to, the resultant S must fall below the point of contact of the toothed rim and the incline, so as to produce a moment forcing the dam downward. If S falls above the point of contact, the moment would cause the dam to rise.

The analysis of these forces can, of course, be made analytically, but the graphical method is better because of its simplicity.

All external forces being known, the roller can now be dimensioned and the materials proportioned. Of course, so far the dead load has only been guessed at and it might be found necessary to adjust the calculations after this load is definitely known.

In the accompanying table are given the weights of some of the rolling dams now in operation. Figuring out the weight in tons per square foot of the projected "wet area" of the rollers (clear width multiplied by the height) it is apparent that this unit weight increases with the area, so that for small rollers it is from 0.035 to 0.045 ton per square foot and for larger from 0.065 to 0.075. In the accompanying curve the total weights of the rollers have been plotted against the projected "wet area" in square feet, so that a designing engineer will be able to get a fairly close value of the dead load as soon as he has determined the "wet area" of the roller. It will be noted that three points on the diagram have not been used in plotting the curve. These points represent the three first roller dams—the Schweinfurt dam of 1902 and 1903 and the Berlin dam of 1904. These are heavy, having no sheath.

In dimensioning the roller two different cases of loading must be investigated—one without any strain on the chain with the roller resting on the sill and one with strain on the chain and no bearing on the sill. In the first case there are two separate conditions—with and without water pressure. The second also has two conditions, one with water pressure and one when the roller is lifted entirely out of the water.



Weights of Rolling Dams

The maximum torsional moment on the roller exists at the beginning of the lift and is expressed by $M_o = Lr$, where M_o is the moment around the cylinder axis and r the radius of the drum at the end of the roller (here assumed equal to the radius of the pitch line for the gear). At the driven end the moment actuating the roller is $M_l = L\ (1 + \sin \alpha)\ r$; α is shown in the diagram.

Rolling-Dam Installations

Locality	River	No	Clear Width, ft. and in.	Height, ft. and in.	Weight, tons	Purpose	Built
			UNITED STATES				
Washington	Spokane	2	100—1	19—0	......	Power	1910
	Long Lake	3	65—0	19—0	......	Power	Constr
Idaho	Boise	1	30—0	8—0	......	Irrigation	1913
Colorado	Grand River	1	60—0	15—4	......	Irrigation	Constr.
		6	70—0	10—3			
			MEXICO				
Torreon	Nazas	2	59—9	8—2	......	Irrigation	1911
Trasquila	Nazas	2	80—5	11—10	......	Irrigation	1910
			CHILE				
La Union		1	85—4	6—7	.....	Irrigation	1910
			AUSTRIA				
Troubek	Bečwa	1	32—10	5—9	18.8	Power	1905
Landeck	Trisanne	1	45—11	3—7	......	Power	1910
Oslawan	Oslawa	1	75—6	4—11	......	....	....
			GERMANY				
Schweinfurt	Main	1	59—1	13—7	72.6	Power	1902
		1	114—10	6—7	87.7		1903
Berlin	Landwehr Canal	1	18—3	5—2	5.6	Navigation	1904
Kolbermoor	Mangfall	1	98—5	5—7	39.0	Power	1905
Bromberg	Brahe	1	72—2	8—2	37.0	Harbor regulation	1905
Kissingen	Saale	1	32—10	8—8	10.2	Power	1906
Nienburg	Bode	2	32—10	8—8	12.0	Agriculture	1906
Neugattersleben	Bode	1	57—5	9—8	24.4		1906
Löderburg	Bode	1	44—6	7—3	......	....	....
Bleckendorf	Bode	2	29—6	8—2	......	....	1910
Hadmersleben	Bode	1	54—2	8—2	......	....	....
Oschersleben	Bode	1	31—6	10—4	......	....	......
Poppenweiler	Neckar	2	91—10	11—10	67.0	Power	1909
Beiertheim	Alb	1	34—5	5—4	......	....	1908
Heimboldshausen	Werra	1	91—10	6—7	......	....	1908
Trostberg	Alz	1	49—3	9—10	......	Power	....
Celle	Fushe	1	49—3	7—7	......	....	1909
Küps	Rodach	1	39—4	5—7	......	....	....
Tübingen	Neckar	2	86—11	7—1	......	....	....
Kibling	Saalach	1	44—7	27—10	......	....	1910
Cassel	Fulda	2	79—9	15—9	......	....	1910
Augsburg	Lech	1	65—7	7—10	......	....	1911
Hamm	Lippe	1	59—1	10—10	......	....	....
Wirtheim	Kinzig	1	49—3	5—11	......	....	....
Fröndenberg	Ruhr	2	82—0	6—7	......	....	....
Plauen	Elster	1	85—4	6—7	......	....	....
Kolin	Elbe	2	62—3	6—4	......	....	....
			FINLAND				
Ensokoskī	Wuochson	2	59—1	11—6	45.1	Power	1907
Nokia	Kumo Älf	2	76—5	6—10	......	Power	1911
		1	68—11	8—6	......	Power	1911
		1	23—0	8—6	......	Power	1911
			FRANCE				
St. Michel	Arc	1	98—5	9—10	62.0	Power	1907
Pontamafrey	Arc	1	108—11	9—0	......	....	....
			HUNGARY				
Zólyombrézó	Garam	1	23—0	3—3	......	....	....
			ITALY				
Verona	Adige	1	39—4	11—2	27.0	Power	1907
Osimo	Potenza	1	32—10	2—8	......	Power	1913
			NORWAY				
Aarlifos	Tinelven	2	49—3	16—5	......	Power	1912
Askim	Vamma	1	91—8	11—6	......	Power	1913
		1	65—7	8—2	......	Power	1913
			SWEDEN				
Trollhättan	Göta Älf	2	65—7	13—1	55.7	Power	1908
Lilla Edet	Göta Älf	2	72—2	12—6	55.0	Power	1911
Dejefors	Klarälfven	2	104—11	11—6	90.0	Power	1908
Porjus	Lule Älf	1	39—4	11—6	19.0	Power	1911
Älfkarleby	Dalälfven	1	65—7	9—2	33.5	Power	Constr.
			SWITZERLAND				
Schönenwerd	Aare		55—9	6—7	20.6	Power	1913

Temperature stresses must also be taken care of. As one side of the roller is generally submerged and thus kept cool, while the other side is subject to the effects of the sun's rays, stresses tending to bend the roller will result. At Schweinfurt, where one roller is 114 ft. 10 in. long and 6 ft. 7 in. in diameter (described in the Engineering Record of March 26, 1904, page 390), it was noted on warm summer days that the temperature stresses arched the roller to such an extent that it was lifted from the sill at the center, causing a leak at that point. As the arching stresses act against the dead load of the roller, which tends to deflect it downward, the resulting curve of the under side of the roller was calculated and the sill, originally a straight line, given the same curvature. When there are no temperature stresses the roller is deflected at its ends due to its dead load, thus affording a complete seal.

The development of these dams is largely to be credited to the Maschinenfabrik Augsburg-Nürnberg, of Germany. Herrn M. Carstanjen, director of the bridge department of that firm, designed the first rolling dam, that at Schweinfurt, in 1902, and the company has since made a specialty of building such dams. The available literature on the subject consists mostly of reprints of papers delivered before engineering congresses and advertising matter from the German firm. These papers contain descriptions of interesting roller dam installations, but not until Professor Hilgard wrote his book on "Bewegliche Wehre" in 1912 was there any text-book information on the subject.

Efficiency in Technical Education

Importance of the Engineering School in the Development of Professional Ideals and Suggestions as to Ways to Make It More Potent

"THE modern need in the field of engineering is for men who can perform the exceptional task; for men who are safe keepers of their brothers' interests; for men whose qualities of character are so sound and strong that they instinctively perceive the way of truth and follow it; for men whose activity and understanding detect the defects in established practice and find a way to improve that practice; for men who so well understand the fundamental principles of science as to be able to predict the effect of the next step before it has been taken; for men, in fine, who can rank with the world's great leaders in science and industry." So spoke Dr. W. F. M. Goss, dean of the College of Engineering of the University of Illinois, in his presidential address delivered at the annual meeting of the American Society of Mechanical Engineers last week. He believes it is the purpose of the technical school to contribute toward the development of such men. In his address, from which these notes are taken, he discusses the technical school, and suggests ways of increasing its efficiency.

Need for Great Teachers

The achievements of the technical school center in the quality and strength of its instructional staff. The present-day need is for increased strength of staff. Great leaders in the work of the technical school are few, and the unfortunate fact is that throughout the years that are past, few men have been in training where the service now demands many There are many reasons for this. An important one is to be found in the limitations which have been put upon the salary budget. The man who builds great bridges or who directs the activities of a great industry is none too gifted to guide the efforts of ambitious youth, and hold before the student large ideals of life. And yet, in the industries a man may receive in a month an amount equivalent to the annual salary of many college professors. It is clear to all who know the field that while the school must suffer for some years to come through its deficiencies in the past, a new policy should be entered upon as speedily as possible. It should be expressed in such terms as will convince the exceptional man, looking for a career, that it will be worth his while to prepare himself by study, by professional practice, by travel, by activities as a scientific investigator, by every process which can develop and broaden, for a life-work as a member of a college faculty.

Not only is it required in the interest of higher efficiency that the American technical school have an instructional staff of the highest possible quality, but it is also required that such a staff be not overburdened with routine duties. The American youth is greatly influenced by the personality of his instructor. While the exceptional student will view his problems broadly, will add his own personality to that which the professor gives, and thus work out large and vigorous conceptions, the normal student is as a disciple following a master! He admires the master's skill, he thinks in terms of the master's thoughts, and is very likely to be influenced by the master's limitations. If the engineering graduate sometimes degenerates into an animated slide rule, may it not be possible that he has been instinctively led to such a career through ill-conceived tread-mill processes in the classroom? If so, the remedy is to be found in reforming the work of the class-room, and one sure road to such a reform is that which opens the way whereby men of large caliber may have time in which to impress themselves upon their students. The technical school is greatly in need of simple living and high thinking. The simple living is easily attained unto, but high thinking, in its essence, involves a certain element of leisure, or perhaps freedom in the choice of one's occupation, which, after all, is but another term for leisure, and no atmosphere surcharged with high thinking can prevail on a campus where every individual student and professor is perpetually keyed up to concert pitch in an endeavor to accomplish an assigned task. This is a matter in which our English and German neighbors can, by the trend of their procedure, disclose much which will be serviceable to the American technical school.

Importance of Superior Leadership

The importance of superior leadership in the school is emphasized by the fact that the attitude of the American student reflects rather faithfully the changing spirit of the times. The danger appears in a disposition on the part of students to segregate their activities, to regard their study as work and their athletics as play. The habit of placing the emphasis continually on the joy of the play prevents him from feeling the joy of the more intellectual pursuits and it encourages half-hearted study. It is here that masterful leadership in the classroom will assert itself. The joy of scholastic achievement is best made plain by the introduction and maintenance of inspiring views depicting the meaning of a life of service coupled with the conception that student days are part of such a life; that student days are not merely a preparation for life, but that they are life.

Masterful leadership eliminates entirely the feeling once entertained by the students that the instructor is a taskmaster, and that to be obedient to him, the student must perform the task. The great teacher, freed from the burden of excessive routine, may easily recognize differences in ability and will encourage the student who must plod and inspire to unusual performance the brilliant student who knows no limit to his achievement save his physical strength.

Value of Extensive Reading

Assuming the technical school to be in possession of an ideal instructional staff, the way will open for progress through many channels of secondary importance. It is altogether possible that in our present-day routine too much time is given to things which are simple. Much that is now studied may perhaps be read. The habit of studying intensely a few books to the entire exclusion of the great mass of historical and biographical engineering literature affords the student but little opportunity of acquiring a habit of rapid and intelligent reading, which in itself is an accomplishment worth striving for. The practice of the shop laboratory, the drawing-room, the surveying field and the study of descriptive texts, rightly interpreted, are important adjuncts in the training of the engineer, but the time has ceased to be when such activities constitute the chief characteristics of the technical school. Year by year the technical school has increased the emphasis given to processes which are mathematical. The progress of the next decade will be seen in the thoroughness with which high standards in such work are accepted and advanced. The intensive work of the course must be based upon fundamental theory, and the fields to which such theory is applied must be broadened. The engineering graduate is no longer required to be prepared to operate machines, but he must have a well-trained mind, and he must possess power to perfect his qualifications along any specialized lines in the shortest possible time. To this end, the years in college must be spent in acquiring an understanding of principles and in the development of those aspects of theory which are difficult to acquire after the completion of one's college course.

Laboratory Possibilities

The American technical school has already accomplished much in the development of laboratories. While it has used them thus far chiefly as facilities for instruction, it has always recognized the fact that there are other and equally important functions to be performed by them. The laboratories of a considerable number of schools have made important contributions to the sum of human knowledge. This fact is suggestive of future possibilities. The practice of men must be guided by facts. The technical school cannot content itself with routine service. It must enshroud its classroom and its laboratories with an atmosphere of scientific achievement; its professors should be leaders not only as classroom instructors, but nation-wide leaders, even world-wide leaders, in the complex and highly diversified fields of the science which they represent. The laboratories available for the use of such men should multiply. Every technical school should aid, as individual schools have already aided, in setting forth new interpretations of physical phenomena. The laboratory must be more than a shop, more than an engine room, more than a collection of testing machines. No great laboratory can be ordered ready-made. It cannot be produced in response to a decree. Whatever its dimensions or its cost, it can be great only in so far as it reflects a purpose which is scientifically sound, and employs means which are scientifically correct. Its significance is necessarily limited to the qualities of the men who create and operate it. A laboratory which has been evolved through the activities and desires of a master is not only priceless for the school that possesses it, but necessary as a source of information of the highest value, to the field of practice which it is designed to serve. Assuming that the country has need of research laboratories for purposes quite apart from the work of instruction, it can, Dr. Goss thinks, be shown that the technical school constitutes the most promising agency in our national economy upon which to place the responsibilities incident to their creation and maintenance.

Work of the Technical School

The relation existing between the standard of instruction in schools which prepare for a profession and the ideals of the profession is obvious. The fact that educational institutions of high standing are sending out each year into the engineering pursuits of our country more than 2500 graduates, nearly 1000 of whom enter the field of mechanical engineering, suggests the basis for this relationship. The technical school is, in fact, recruiting and otherwise stimulating the engineering work of the country. Its more important contributions may be summarized as follows:

First, the work of the school tends to emphasize the dignity of the calling. Professionalism as distinguished from the art of practice of engineering is after all a question of quality. Professionalism is breadth! In our ambition to have a large part in the world's affairs we sometimes forget the fundamental source of power. We sometimes think too much of honors and too little of service; we desire the position before we are qualified to discharge the responsibilities it imposes. Our strivings must be for fitness. Progress in advancing the ideals of our profession will depend upon the character of the service rendered by the members of the profession. If every practitioner in the field of mechanical engineering possessed character and ability responding to our highest ideals, no one would question the right of mechanical engineers to regard themselves members of a profession. The technical school makes its contribution to the upbuilding of professional ideals by sending trained men into the profession.

Second, a service which the technical school is rendering the profession of engineering is that of contributing to the sum of its scientific data. Engineering as a science has made progress by leaps and bounds; rule of the thumb has given way to the rule of the mathematician and investigator. But notwithstanding all that has been accomplished in this direction, there are as yet but few departments in the field of mechanical engineering in which the basis of design is actually perfected. Further progress awaits the establishment of facts concerning the behavior of constructive materials, of lubricating films, of combustible mixtures, and of liquids and gases employed in thermo-dynamic processes as vehicles for the transmission of heat. The American engineer who has hitherto been occupied with the so-called practical aspects of his profession has concerned himself less than have the engineers of the great empire across the sea, with the work of the scientist. We, as engineers, need to train ourselves to a condition of mind which will make studious processes less difficult than at present. Along this line of endeavor the researches of the technical school will aid the profession by establishing new standards of proficiency, and by making new contributions to the existing fabric of facts.

Third, an important contribution which the school is rendering the profession of engineering is that of emphasizing the profession's unity of purpose. The college graduate has had at least four years' training in teamwork. A majority of the student organizations with which he has been associated has maintained extra-mural affiliations.

Whatever may be the nature of the contribution of the technical school to the ideals of the profession, the extent of the contribution is a function of the quality of the school. Better instruction in the school must supply better recruits for the profession, greater activities in scientific researches on the part of the school must operate to increase the facility with which the problems of the engineer are solved, and influences in the school which tend to extend the student's horizon and broaden his sympathies will in due time make their impress upon the professional life outside of the school. The problems of the technical school should not be left to the schoolmaster, for, broadly interpreted, they are not the problems of the school but the problems of the profession.

Proposed Restrictions for Heights of Buildings in New York

Commission Submits Recommendations for Dividing City into Districts and Controlling Character and Heights of All Structures in Each District

"WHEN high buildings are crowded together the result is mutually disastrous to all owners. Artificial light must be used on the lower floors even on the brightest day in summer. The darker offices usually rent for much less than those with better light. When such conditions prevail it is clear that a reasonable restriction on height and court area, if applied at the start, would have been greatly to the advantage of all owners concerned." These are some of the conclusions which the Heights of Buildings Commission of New York City has incorporated in its report submitted on Dec. 3 to the Board of Estimate and Apportionment. The restrictions recommended limit the height of a building at the street line to twice the width of the street, but this limit in no case is to be less than 100 ft. nor more than 300 ft. After reaching the limiting elevation the structure may be carried higher by setting the street walls back 1 ft. for each 4 ft. of increased height. The report also recommends that the city be divided into building districts and that to buildings erected in these districts certain restrictions as to height, character and use be applied. Mr. George McAneny is chairman of the Board of Estimate's heights of buildings committee and of the advisory commission Mr. Edward M. Bassett is chairman and Mr. George B. Ford, secretary. The report, in part, follows:

The commission finds conclusive evidence of the need of greater public control over building development. The present almost unrestricted power to build to any height, over any proportion of the lot, for any desired use and in any part of the city has resulted in injury to real estate and business interests, and to the health, safety and general welfare of the city.

General Restrictions for All Buildings

A building of excessive height is not necessary in order to realize the maximum net return from the land. The tallest buildings often do not pay the best. The entrance hall, elevators, stairs and services take too much valuable space. Even supposing the building can be advantageously rented at the start, its prosperity will probably continue only so long as it is not surrounded by buildings of similar height.

Lack of sunlight and the continuous use of artificial light undoubtedly have a direct relation to health, eyestrain, and general physical and mental efficiency. The health and comfort of the hundreds of thousands of office employees are matters of great public importance. Equally injurious is the lack of adequate ventilation due to the opening of work rooms on deep and narrow courts within which any circulation or renewal of air is difficult. The street subsurface is becoming overcrowded with sewers, pipes, wires and rapid-transit subways—all occasioned in considerable measure by the extreme heights of buildings.

The restrictions recommended are designed to secure as much light and air, relief from congestion and safety from fire as is practicable under existing conditions as to improvements and land values. In place of proving a menace to existing values they will tend to prevent future serious injury to such values.

The proposed regulations are in full as follows:

Street Walls

1. Except as hereinafter provided, when the street walls of any building reach a height equal to twice the width of the street they shall be set back from the street in the ratio of 1 ft. horizontally for each 4 ft. vertically, but the street walls of a building facing on any street, public place, park, or body of water, more than 150 ft. wide, including an intervening street, if any, must begin their setbacks not more than 300 ft. above the curb, except as hereinafter provided for towers. Street walls if erected on the building line may reach the height of 100 ft. on a street less than 50 ft. wide before the setback as stated above must begin. When the width of a street varies in a given block, the width of the street for the purpose of determining the height of the street walls in said block shall be taken to mean the average width of said street in said block.

2. When street walls are erected inside the building line so that a space intervenes between the street and the wall, the setback shall begin where such wall intersects the setback plane as determined by the setbacks in paragraph 1, and above that point the wall shall set back in the same manner as if the wall were placed on the building line.

3. Where a single building is erected upon a corner lot facing upon streets of different widths, the street of greatest width may be used to determine the height at which the setback shall begin. The mean level of the curb in such street of greatest width shall be the point from which such height shall be measured.

3. Where a single building not on a corner lot abuts upon streets of different widths the height and setbacks of each street wall shall be determined by the width of the street on which it abuts.

5. No cornice shall project more than 5 per cent of the width of the street beyond the building line or the plane determined by the required setbacks in paragraph 1.

Courts

6. Every building may cover the entire area of the lot up to and including the tier of beams forming the ceiling of the first story, which shall be that story the floor of which shall be not more than 7 ft. above the curb level at the highest point of any street on which the building abuts.

7. Except as hereinafter provided, on all lots upon which buildings shall be erected provision for light and air shall be made by leaving yards or uncovered courts above the second-story floor level, whose least dimension shall be not less than 6 ft.

8. At any story of a building the least dimension of any court, measured to an opposite wall of the same building or to a lot line, shall equal in feet at least one and one-quarter times the number of stories from the second floor to and including said story. This provision need not apply to a rear yard as required under paragraph 12.

9. In a court of irregular shape the least dimension shall be taken to mean the least distance between walls or between any wall and a lot line measured on a line erected perpen-

dicular to the center of any side of said court.

10. The provisions of paragraphs 7, 8 and 9 need not apply to a court upon which no office or work room solely depends for access to outside light and air.

11. In every building there shall be a loss in area for each story above the second story floor level of at least 1 per cent of the lot area, in addition to other requirements hereinafter contained.

12. Except as provided in paragraph 13, there shall be an uncovered space above the second-story floor level between the rear line of every building and the rear line of the lot, which shall contain not less than 10 per cent of the area of the lot and the least dimension of which shall be not less than 10 per cent of the depth of the lot. When the front and rear lines of the lot are not parallel, the depth of the lot shall be taken to mean the average depth.

13. The requirements of paragraph 12 shall not apply to a building erected on a lot at the corner of two or more streets. When a building is erected upon a lot fronting upon two or more streets but not on a corner there shall be an uncovered space above the second-story floor level equal in area to 10 per cent of the area of the lot.

COURTS

14. No courts shall be required in a building erected on a three-sided lot in which three sides face upon public streets and in which the length of the shortest side does not exceed 100 ft.

15. No courts shall be required in a building erected upon a three-sided lot in which only two sides face upon public streets and in which the length of the third side does not exceed 100 ft.

16. No courts shall be required in a building erected upon a rectangular or trapezoidal lot in which three or more sides face upon public streets, and in which the greatest width of the lot from street to street measured in a line at right angles to either street does not exceed 90 ft.

17. It is further provided that, in addition to a building erected as hereinbefore provided, a structure to be called a "tower" may extend without limit above such building and without loss of area, but such tower shall not occupy an area exceeding 25 per cent of the area of the lot, and no part of such tower shall approach nearer than 20 ft. to any lot or street line, except, however, that such tower may be built on that building line of a building facing on a public square, a public park, or the waterfront, with or without an intervening street, as defined in paragraph 1.

18. The above regulations do not apply to tenement houses and do not apply to hotels in so far as paragraphs 6 to 16 in relation to courts are concerned, nor do they apply to church spires, belfries or chimneys for power and manufacturing plants. The existing laws and ordinances in relation to tenement houses and hotels will be continued in force.

HEIGHT REGULATION DISTRICTS

The city should be divided into districts and the restrictions for each district worked out with reference to the peculiar needs and requirements of that particular district. The blanket restrictions recommended for immediate adoption have, as a matter of fact, been devised with reference to the needs of the downtown office and financial district—the area of maximum congestion.

Every city becomes divided into more or less clearly defined districts of different occupation, use and type of building construction. We have the central office and financial district, loft districts, waterfront and industrial districts, retail-business districts, apartment-house and hotel districts, tenement-house districts and private-dwelling districts. A comparatively high degree of concentration is believed to be important for the facilitation of business in the office and financial district. These natural districts must be recognized in any complete and generally effective system of building restriction. The restrictions should be based on the theory that assuming that the entire district should be built up uniformly with buildings of the maximum height and extent allowed, the provision for light and air would be adequate, and the district as a whole would be appropriately improved. While the commission knows of no immediate practicable remedy for the existing congestion of population on the lower East Side, it believes that by appropriate restrictions, varying with the district, repetition of these conditions in other parts of the city can be prevented. A few comparatively small districts of the city are already spoiled, but most of the area of the city is still in condition to be greatly helped by appropriate regulations.

The commission recommends that the Board of Estimate and Apportionment be empowered by the State Legislature to district the city for the purposes of building-height and court-area restrictions and to apply to buildings hereafter constructed different restrictions in different districts. It is recommended that the Board of Estimate and Apportionment, upon receiving such legislative authorization, appoint a commission, which commission after hearings shall recommend to the board the precise boundaries of the several districts, and the regulations to be applied in each such district.

EIGHT CLASSES OF DISTRICT

While the Commission does not specify the exact number of districts to be created, or the precise restrictions, certain tentative conclusions are presented merely by way of suggestion and illustration. The following eight classes of district should be provided for:

A Districts.—General restrictions recommended for immediate adoption, regulating heights of all buildings.

B Districts.—Twice the street width, and not over 150 ft. Setback 1 ft. horizontally for each 2 ft. vertically.

C Districts.—Twice the street width, and not over 125 ft. Setback same as B.

D Districts.—One and one-half times the street width, and not over 125 ft. Setback 1 ft. horizontally for each 1½ ft. vertically.

E Districts.—One and one-half times the street width, and not over 90 ft. Setback same as D.

F Districts.—Once the street width and not over 80 ft. Setback 1 ft. horizontally for each 1 ft. vertically.

G Districts.—Not over 50 ft. Setback same as F.

H Districts.—Not over 36 ft. Setback same as F.

When the street front of any building shall have reached the height limitation, the building may still be erected to a further height at a point set back from the street to the distance provided by the setback regulations. The setback regulations are to be understood to permit vertical walls or pitched roofs or other structures, provided only no part of such structure rising above the height limited at the front wall shall extend above the limit allowed by the particular setback provision. Where the height limit is the street width or a multiple thereof the setback provision is designed to preserve a certain angle of light determined for the various classes of districts as herein set forth.

The aforementioned eight classes of districts were worked out after a careful study of land values and improvements throughout the city. It seemed that every portion of the city could be appropriately placed in some one of these eight classes without sacrificing existing values.

Class A restrictions, under the districting plan, should be confined to the area or areas of maximum business congestion, namely, much of the lower end of Manhattan below Park Place, Broadway to Fifty-ninth Street, and certain limited areas south of Forty-second Street. Classes B, C, D and E are designed to cover most business and industrial districts and also high class hotel and apartment house districts. Most of Manhattan, small portions of Brooklyn, Queens and The Bronx and no part of Richmond should be included in these districts.

It seems that a very small portion of Richmond and Manhattan and very large portions of Brooklyn, Queens and The Bronx should be included as F and G districts.

Class H restrictions limit height at building line to 36 ft. This will mean for the most part the building of one and two-family houses, and should be applied to districts where this type of construction is most appropriate. It seems that almost all of Richmond, most of Queens and large areas in Brooklyn and The Bronx can appropriately be included as H districts.

In the illustrations and suggestions in regard to districting the commission has made no reference to restrictions as to courts and yards, save those contained in the recommendations for District A, which would prevail for the entire city unless superseded. This is a difficult subject and in working it out it is possible that it would be desirable to increase the number of classes of districts. It may, for example, be desirable to divide class H, where the 36-ft. height limit obtains, into two or more classes, with different limitations as to courts and yards. Provision for adequate courts and yards is of the utmost importance in carrying out a well-considered plan of building development.

RESIDENCES AND FACTORIES

Height limitations alone will not prevent deterioration of sections owing to the invasion of inappropriate industries or structures. When single-family dwellings, apartment houses, stores and factories are thrown together indiscriminately the health and comfort of home life are destroyed and property and rental values are reduced.

Factories and other industries should not be permitted to enter certain residence sections. In certain districts a man should be able to build a home in a neighborhood of his choice without the hazard that in a few years through the building of apartments or other structures the location will become undesirable for a home of the character he has built and his property will be seriously depreciated. Reasonable restrictions will tend to stabilize existing districts. The commission recommends that an act be passed by the State Legislature authorizing the Board of Estimate and Apportionment to regulate the location of industries and the location of buildings designed for specified uses and to establish districts for this purpose.

AN IRRIGATION DAM to impound 7500 acre-ft. of water is in course of construction near Lliu Lliu in Chile, according to a recent consular report. The dam will be 125 ft. high and 2500 ft. long at its crest.

Safeguarding Displaced Viaduct Piers

Stopping Dangerous Settlement and Sliding by Drainage System; Underpinning and Rebuilding Foundations at Cleveland

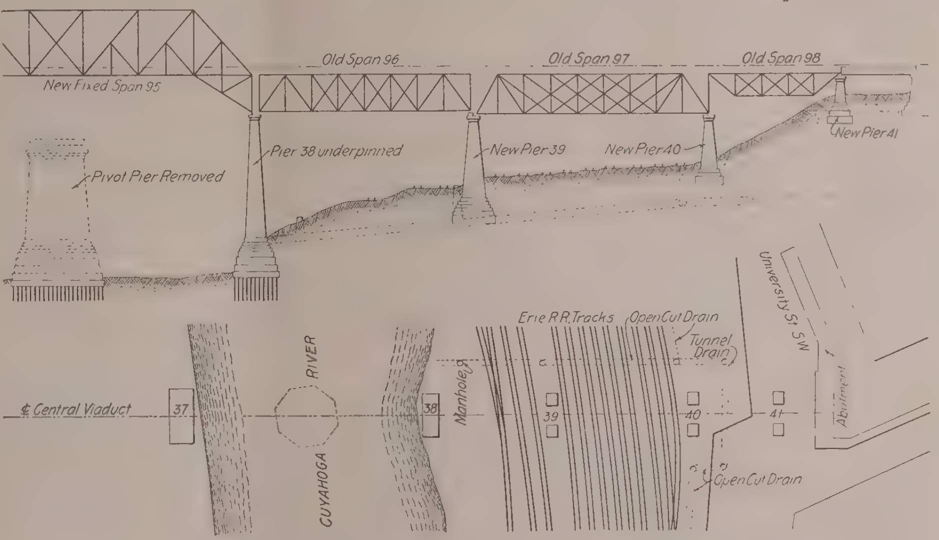
THE steel highway viaduct across the Cuyahoga River and valley and a number of railroad tracks in Cleveland, Ohio, has tall masonry piers on earth and pile foundations. Movements have been observed in some of them ever since their construction in 1887. Excessive settlement and large displacement parallel to the bridge axis eventually injured some of the piers, displaced the superstructure so much that portions of it had to be cut off, and endangered the safety of the viaduct. In order to provide for greatly increased traffic and to insure the safety of the structure for a few years until the whole is replaced by a new one some of the piers have been rebuilt and others have been reinforced with additional foundations, involving a considerable amount of difficult and dangerous work of a somewhat unusual character, which has been successfully completed.

The viaduct is about 2835 ft. long, 100 ft. high and 56 ft. wide, with two electric-car tracks, two roadways and two sidewalks. The 236-ft. swingspan over the river was replaced last year by a fixed span, as described in the Engineering Record of July 6, 1912, page 27, which eliminated the pivot pier and provided increased headway sufficient for the navigation requirements, thus saving in operating expenses and greatly facilitating traffic. The new span is supported on the old rest piers (37 and 38) for the swingspan, and in order to carry the increased loads their foundations were extended and the old masonry preserved, thus effecting the saving of a large portion of the $150,000 which it was estimated would have been the cost of new piers.

MOVEMENT OF PIERS

Movements of piers 38 and 39 were noticed when they were built in 1887, but observations on the other piers were not recorded at that time. In 1894 a new base line was established and the observations from it were made on all piers. As these were not tied up to the old line, the movements occurring between the last observation on the old line and the first on the new line were not definitely ascertained. From later observations it was found that the movements were continuing in piers 38, 39 and 40, but none was observed in pier 41 or the adjacent abutment. The movements are shown on the accompanying diagrams. The horizontal movements toward the river attained ultimate amounts of 1.82 and 1.11 ft. for the east and west shafts of pier 38, 1.94 and 1.88 ft. for the shafts of pier 39, and 1.11 and 0.75 ft. for the shafts of pier 40. Piers 38 and 39 have also moved downstream, transverse to the bridge axis, 0.65 and 0.75 ft. respectively.

As all of these movements were determined by measurements made from the base lines to marks on the copings, it was at first uncertain whether the piers were moving bodily or simply being revolved about their bases. To determine this point marks were accurately located both at the coping and near the ground on the side center line of each pier, and the lines thus determined being plotted on the original plans and produced to intersections with the bases of the piers, it was found that pier 40 had rotated about its base, pier 38 had moved bodily without rotation, and pier 39 had moved bodily and rotated. From a study of these movements it was concluded that they were due to a landslide commencing somewhere between piers 39 and 40.



Location of Displaced and Settled Piers and Sidehill Drain

Borings indicated that the land piers were founded on stiff clay with considerable water and some quicksand above the clay, but they did not disclose well-defined strata of quicksand or water-bearing materials. Nine tracks of the Erie Railroad Company, transverse to the bridge axis, were in operation between piers 39 and 40, and three north of pier 39, and it was thought by some that the vibrations caused by these heavily loaded trains in the saturated soil of the side hill were sufficient to produce slipping on the inclined clay strata. In view of this opinion it was believed that drainage of the soil might stop the movements, and work was undertaken for this purpose before the pier repairs were commenced.

In 1911 a drain about 500 ft. long was built parallel with and close to the uphill side of the group of tracks passing under the bridge between piers 40 and 41 and extending for some distance beyond it on both sides, and an outlet was provided for it by means of a branch about 250 ft. long connected to it at a point near the center and running parallel with the bridge axis to its outlet in the river. Part of the drain consisted of a 3 ft. 8 in. x 4 ft. 8 in. two-ring-brick egg-shape section built in tunnel under the railway tracks and bridge, and the remainder consisted of a three-ring circular section 24 in. in diameter inside, which was built in open trench about 24 ft. deep.

The entire tunnel excavation outside the brickwork was filled solid with masonry to

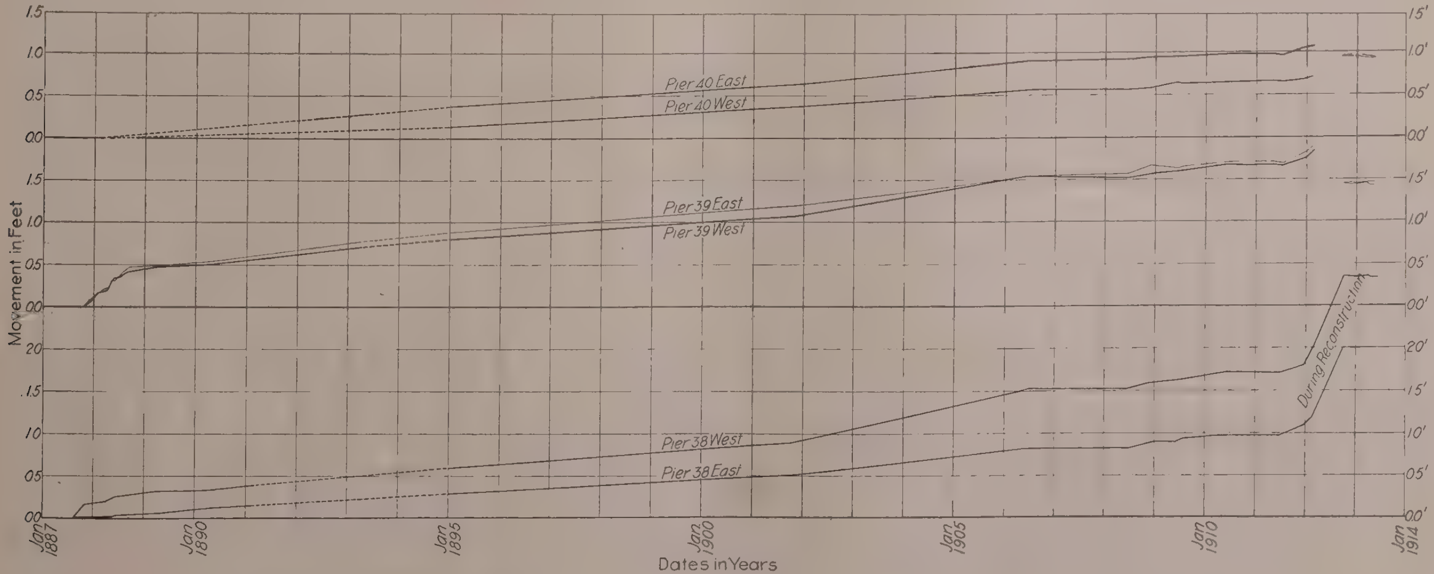
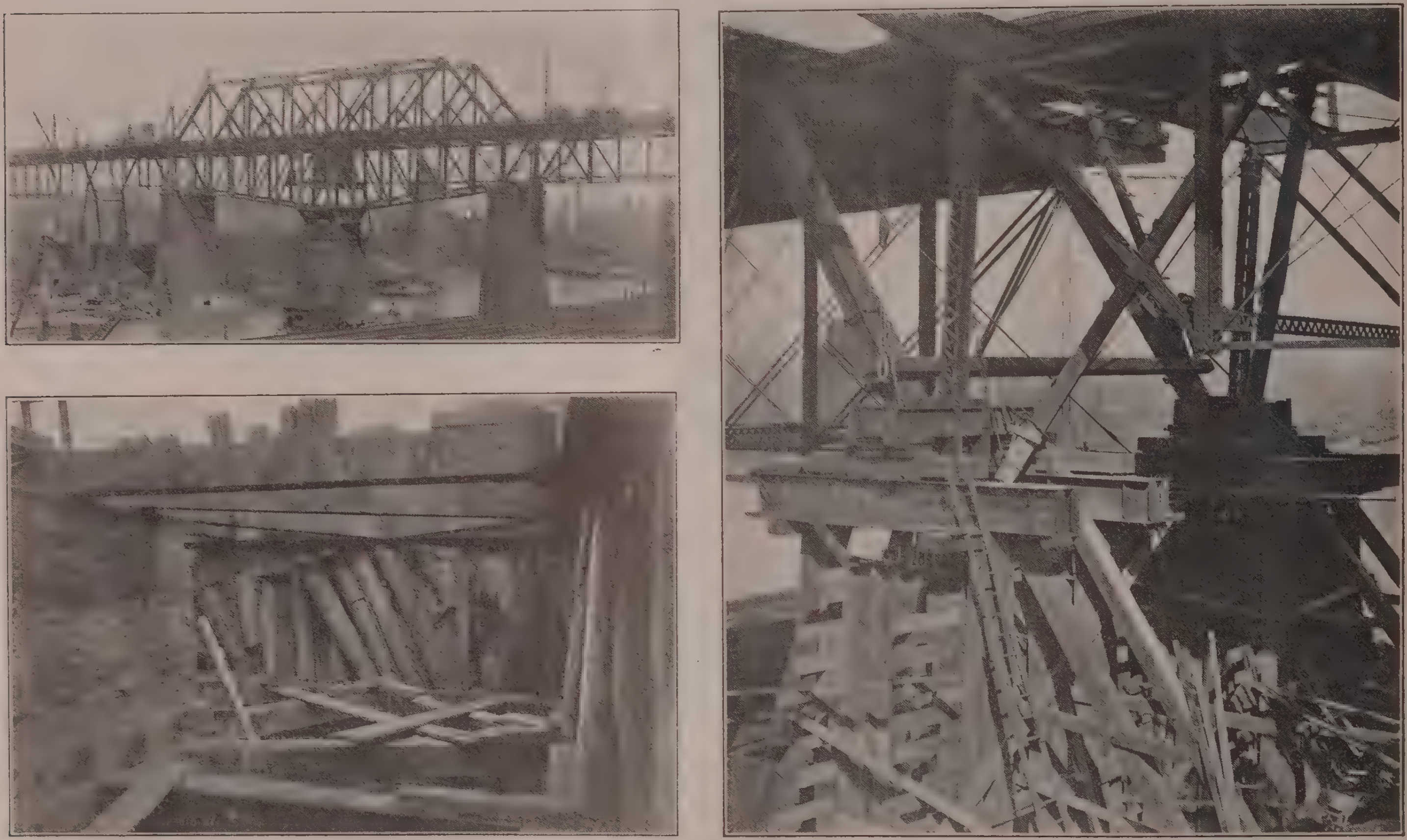


Diagram Indicating Settlements in Twenty-six Years for Both Shafts of Each of Three Piers guard against settlement of the tracks. The circular section invert was built with full mortar joints, but the bricks in the arch were only bedded in mortar, the mortar being omitted in the end joints. Open butt joints were also left in the arch every 3 ft. from spring line to spring line, and the arch was covered with a layer of straw and the trench backfilled with cinders to provide for percolation into the drain.

Before sufficient time had elapsed to determine the effect produced by the drainage, appropriations were made for the repair of the viaduct, and the work here described was carried out. No displacement whatever has since been observed in the piers, and it is not yet determined whether the drainage system or the repairs, or both, were responsible for the cessation of movement.



Old and New Channel Spans and Old Piers (Upper Left); Inclined Positions of Old Foundation Piles (Lower Left), and Spans Supported on Falsework Tower for Pier Renewal

EFFECTS OF MOVEMENTS

Between piers 38 and 40 the average longitudinal movement of the viaduct toward the river averaged 1 in. per year, necessitating the frequent amputation of one end of the swingspan and span 96, and it became eventually impossible to revolve the swingspan end for end. Piers 39 and 40 were considerably inclined and cracked and the vertical posts supporting the river end of span 38 were carried 14 in. out of plumb. In 1903 pier 39 was reinforced by heavy timbers and in 1910 pier 40 was similarly strengthened by horizontal yokes securely clamping the framework of vertical timbers against the masonry.

Meantime the constant increase in live loads to which the bridge was subjected, together with the inadequate maintenance provided for it, made it desirable to replace the entire structure within the next ten or fifteen years, and the repairs necessary for present safety and the immediate future were made of a temporary nature, as here described, and with a view to provide a little experimental investigation which would determine the best method of stopping the hill slide so that the new structure could be properly designed. The reconstruction of the river span was not considered necessary, but was advantageous on account of the additional safety and the reduction of operating expense secured. Piers 39 and 40 were in such bad condition that immediate rebuilding was necessary.

UNDERPINNING PIER 37

The foundation of the north river pier 37 was extended as illustrated in the Engineering Record of June 29, 1912, page 716, at a cost of about $28,400. The pier was inclosed by a double-pile and puddle cofferdam, which was unwatered, allowing the timber grillage footing to be undercut about 6 ft. on all sides. The tops of the old foundation piles exposed were inclosed in a solid mass of reinforced concrete extending about 6½ ft. beyond the old footing, where it was supported on about one hundred and fifty new foundation piles and acted as a continuous girder to distribute the load of the pier between the old and new piles and develop the resistance of the latter against further settlement of the masonry. After the completion of the foundation work the cofferdam was removed and the excavation backfilled around the pier foundation.

UNDERPINNING PIER 38

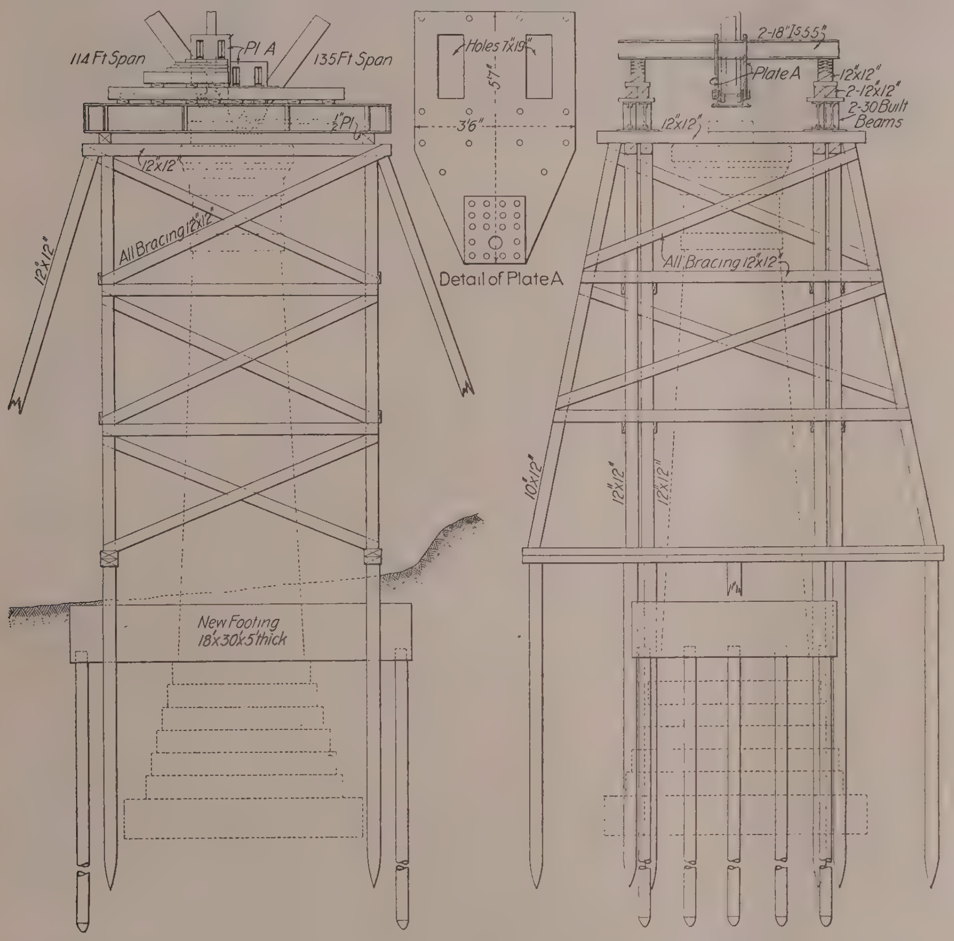
Pier 38, which had supported the south end of the drawspan and now supports the end of the fixed river span, was underpinned in substantially the same manner as pier 37, but on account of the very large movements which occurred previous to and during the underpinning, amounting to a final total displacement of nearly 3 ft., the work was very hazardous, and was very cautiously executed. The concrete extension to the old footing was carried out over the full area of the cofferdam, and the latter was left permanently in position. The old foundation piles were only 30 ft. long, and the excavation, 10 ft. deep around their upper ends, which was necessary for placing the new concrete, reduced their penetration to 22 ft. and still further increased the dangers of the situation. The old foundation piles were found leaning 3 ft. toward the river, and as far as could be seen only about one-third of them were bearing against the foundation grillage. The old protection and cofferdam piles encountered were found inclined toward the river as much as 45 deg. The installation of the new foundation piles gave the most concern, as the driving of each one caused a perceptible movement in the pier. The total movement due to the driving of new piles was 0.19 ft. at the top and 0.61 ft. at the bottom of the pier at the east end and 1.07 ft. at the top and 0.71 ft. at the bottom at the west end of the pier. Constant observations of the pier were made to detect any sudden or unusual movement, and the concrete work was pushed as vigorously as possible, but was seriously retarded by the extremely cold weather.

REBUILDING THE SHORE PIERS

The old masonry in piers 39 and 40 was removed to a few feet below the surface of the ground and the piers rebuilt of 1:2:4 Portland cement reinforced concrete. The concrete was machine-mixed on the bridge floor and chuted to place and the old masonry removed by a derrick, also seated on the bridge floor.

During the removal of the old piers 39 and 40 and the building of the new, the ends of old spans 96, 97 and 98 which they support were temporarily carried on wooden falsework towers inclosing the piers and permitting the old masonry to be removed and the new masonry to be built inside of them. The end lower chord pins of the trusses in these spans were suspended from the falsework towers through pairs of large steel plates which they engaged. The plates were pierced with large rectangular holes engaging transverse horizontal I-beams about 20 ft. long that cleared above the inclined end posts of the trusses. These beams projected beyond the sides of the twin shafts forming the pier and were carried on plate girders in the upper parts of the falsework towers.

The extra pin bearings necessary for the engagement of the hanger plates outside ot the truss bearings were provided by the expedient of removing the original Lomas nuts and utilizing the shouldered ends of the pins, which they had inclosed, for the bearings. The nuts were then replaced in reverse position while the span was suspended. After the hanger plates, beams and girders were assembled in position jacks were placed under the transverse beams and operated to lift the span clear of the bearings on the old piers and lowered them to bearing on the falsework.



Falsework Tower Supporting Spans While Piers Were Rebuilt

Underpinning of Pier 39

As the foundation load on pier 39 did not exceed 4000 lb. per square foot on the stiff blue clay, no additional bearing for it was at first contemplated. During the operations on pier 38 a large fissure transverse to the bridge axis was developed in the ground just beyond pier 39, and in view of this serious lateral movement of the earth it was deemed advisable to enlarge the bearing area of the pier footing. Close to the long sides of the pier five 12-in. wrought-iron pipes, 30 ft. long, were driven on each side to clear the falsework-tower piles and were excavated, filled with concrete, and capped with reinforced-concrete slabs 4 ft. thick, which were made continuous across the old footings on which they were placed after the upper portion of the masonry had been removed.

The pipes were driven by a machine located on the surface of the ground and having a drop hammer which fell 4 or 5 ft., and which was operated by an engine on the bridge floor above. As this work was being executed at the same time as that on adjacent pier 38, great care was exercised in driving the pipes to avoid excessive vibration that might be transmitted to pier 38 before the concrete there had set.

The cost of rebuilding piers 39 and 40 and the incidental work was $14,734.98. The cost of work on the river piers 37 and 38 and for the removal of the pivot pier was $88,811.53. The total cost, including the drainage system, was $119,372.76.

Movements of Spans

Piers 38 and 39 had moved downstream transversely to the bridge axis, displacing the alignment of the superstructure, which after the pier repairs were completed was moved back in the opposite direction to the original position. This involved a transverse movement of 7½ in. on pier 38 and 9 in. on pier 39, which was effected by jacks. The movement parallel to the axis of the bridge required a longitudinal replacement of 3 in. for span 96 and some movement for span 97, both of which were effected by steamboat ratchets.

The work was designed and executed under the direction of the Division of Engineering of the Department of Public Service of the City of Cleveland, of which Mr. W. C. Springhorn is director, Mr. Robert Hoffman chief engineer, and Mr. F. D. Richards bridge engineer. The Great Lakes Dredge & Dock Company was the contractor for underpinning the piers.

Aeroplane for Patrolling Electric Transmission Lines

Western Power Company Employs Aviator to Make Regular Trips of Inspection and Repair

An aeroplane for patrolling long-distance electric-transmission lines has been placed in service by Mr. E. W. Beardsley, general superintendent of the Great Western Power Company, of San Francisco. This is the first time that an aeroplane has been used for such work. When a break in the line is observed the machine can be landed and a lineman who accompanies the aviator can make the necessary repairs. Large numbers have been placed on all of the company's towers in order that they may be easily distinguished from above. The towers are about 750 ft. apart, so that in flying at the rate of 40 miles per hour considerable time is allowed for the inspection of each section of the line.

The Great Western company's transmission line extends from Big Bend on the Feather River, across sloughs and marshes of the Suisun Basin and finally over the Coast Range to Oakland, a distance of 160 miles. At present nearly all of this line must be patrolled on foot, except in the level valleys. Portions of the marshes are impassable and great difficulty is experienced and frequent delays caused when breaks occur in these inaccessible locations. It is hoped by the use of the aeroplane greatly to expedite repair work and thus afford better service to the consumer.

The aeroplane used by the Great Western company is a Wright model, slightly modified by altering the pitch of the propeller so that a speed of 40 miles per hour may be constantly maintained. The wheels and axles on the landing mechanism have been strengthened to permit landing on fairly rough ground. A rigging for the lineman's kit of tools has been attached to the plane, as it is the company's intention to have linemen repair any minor troubles whenever possible. In case of breaks which cannot be handled by one man, he will phone the dispatcher by means of a portable telephone. The dispatcher can send as many men as may be required, with the least possible loss of time, to the exact tower designated.

The aviator selected for the patrol duty is Robert Fowler. He is stationed at Sacramento. The trips, which will be made twice a week, were begun on Nov. 20.

Moving Loads on Steel Bridges

Consideration of Stresses Produced by Train Crossing Truss and Girder Spans

Investigations of the stresses produced in steel railroad spans by locomotives and trains moving across them, at different speeds, have been in progress at Washington University since 1906, and a review of them recently published by Mr. E. O. Sweetser, assistant professor of civil engineering, does not warrant final conclusions, but indicates that impact stresses increase with the speed of the locomotive, but do not appear to have any fixed relation to it for short spans.

Impact Investigations

In 1906 records were made by students on stresses in a 150-ft. pin-connected single track Pratt truss span built in 1883 with suspended floorbeams and light lateral and floor systems. The investigators concluded that "the present formulæ seem to be extravagantly generous in the allowance for impact." In the light of subsequent investigation Professor

Sweetser believed this to indicate something radically wrong in the assumptions made as to the distribution of static stress.

Additional experiments on the same bridge in 1907 lead to the conclusions that for "speeds up to 15 miles per hour there is no increase of stress; for speeds up to 36 miles per hour there is an increase of 25 per cent, and for speeds up to 45 miles per hour there is an increase of 30 per cent in stress."

THREE BRIDGES TESTED

In 1910 and 1911 tests were made on three bridges under the direction of Professor Sweetser. The records were made both under regular traffic and under special trains consisting of a freight locomotive followed by four or five heavy cars run across the bridge at different speeds. The stresses were indicated by special extensometers recording the elongation in a measured distance of 50 in. These two instruments magnified elongations about 56.5 and 59.5 times and recorded the amounts by a pencil moving on paper transverse to the direction in which the paper was moved automatically by clockwork. The speed of trains was determined by timing them with a stop watch over measured distances of 500 ft. for high speed and from 100 to 200 ft. for low speeds.

About 300 records were made of the effects of trains passing over three 153 ft. 1½-in. riveted through-truss single-track spans of bridge 18.2 of the St. Louis & San Francisco Railway at speeds varying from 2½ to 50 miles per hour. This bridge replaced the one previously tested, and had heavy members capable of resisting both compression and tension stresses. Most of the records were made on the center span. The lower chords, main diagonals, one suspender, top chords, secondary verticals, and end posts were tested.

About 50 records were taken of train loads on the single track deck-plate girders, 69 in. deep, in a two span bridge for the Missouri Pacific Railway. The extensometer was placed on the tension flange of the girder at the center of the span. The speed of the trains varied from 7 to 51 miles per hour.

About 200 records were made of train loads on the 36-in. plate girders for a 33-ft. deck single-track span of the St. Louis & San Francisco Railroad, the extensometers being placed on both inner and outer flanges.

About 300 records were made of trains on two spans of 52-ft. deck plate girders 69 in. deep with a track curved 2 deg.

CONCLUSIONS

The analysis of the results of the tests and the curves from them indicate that the distribution of stress in plate girder flanges is not at all uniform and that considerable impact often exists for short spans at speeds of 10 miles per hour, and that such a speed does not necessarily give a static stress.

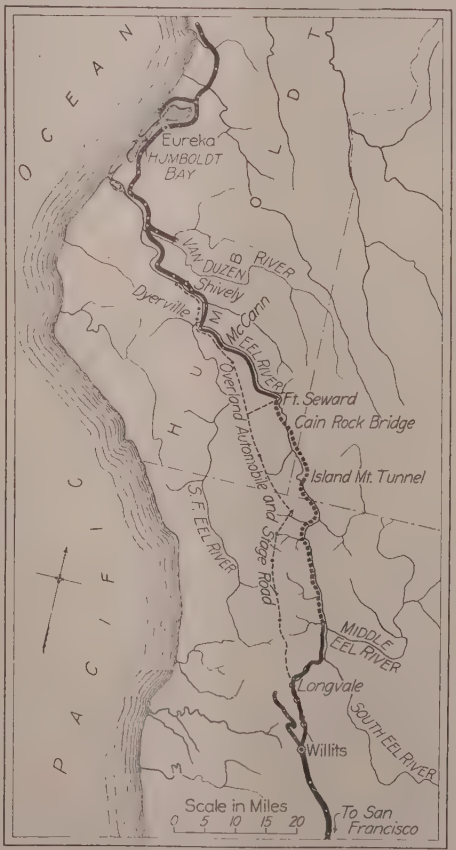
THE ELECTRICALLY PROPELLED COLLIER "JUPITER" has performed a successful forty-eight hour trial at sea off Santa Barbara, Cal. It maintained an average speed of 14.78 knots, or about ¾ knot above the required speed. The "Jupiter" is a 20,000-ton ship, equipped with twin propellers directly connected to three-phase induction motors supplied by a turbo-generator. This ship was equipped with electric propulsion in order that it might be compared with the "Cyclops," which has reciprocating engines, and the "Neptune," which is equipped with geared turbines. All are of the same type and of similar displacement.

Earth Slide Burying Steam Shovel and Work-Train Equipment

Northwestern Pacific Railroad Extension

Closing the 100-Mile Gap between San Francisco and Eureka by Means of a Line through the Rugged Eel River Valley

COMPLETION of the 100-mile link between the two ends of the Northwestern Pacific Railroad in California is now in sight, as more than half the mileage has already been turned over to the operating department. The link is located entirely in the valley of Eel River, which it follows closely on a water grade. Rich agricultural and grazing land and heavily wooded areas, together with one of the most equable climates in the world, make bright the prospects for heavy freight and tourist traffic between San Francisco and Eureka, the largest coast city in California north of San Francisco. Eureka is a town of 12,000 people, located on Humboldt Bay, a deep-water harbor on which the Federal Government has spent $2,000,000 for breakwaters and is now spending $1,000,000 more for repairs.



Map Showing Location of Closing Link

For many years the southern section of the road, which parallels the coast for more than 100 miles, ended at Willits, 8 miles beyond the summit separating the Russian River running south and a tributary of the Eel River running north. At the north end a lumber road, when acquired several years ago, stopped at Shively. As extended south from this point the line traverses 10 miles of sequoia forests of great stately trees to the Dyerville crossing of Eel River and continues on the opposite side to McCanns, where the steam and wagon roads separate, the former following the river closely and the latter taking the crest of the comparatively narrow ridge separating the two forks of Eel River. Track is now laid to Fort Seward.

CHARACTERISTICS OF COUNTRY

This stretch is typical of the whole line, with its sliding earth slopes alternating with steep, rocky hillsides and wooded flats. It is described thus: "The scenery is of a rough, wild type, mountainous, characterized by steep hillsides, precipitous cliffs and great rock masses, many standing, sentinel-like, at the water's edge, or rising high from midstream. Often there are long stretches where the hills rise abruptly from the water, and in many places the river beach is covered with boulders and rocks. Upheavals some time in the past are evident everywhere in broken strata, yet hardy growths of trees and underbrush beautify all spots with their verdure. So narrow is the river valley in most cases for a distance of 50 or 60 miles that human habitations are almost out of the question, although back over the ridge are other valleys larger and richer in soil. In this section the engineers have had to tunnel so many times that if the bores were located at regular intervals there would be one every 7 or 8 miles from San Francisco to Eureka." Where redwoods of value were encountered they were felled parallel to the track, cut to logging sizes, rolled to one side and sold or reserved. The stumps were blown out with dynamite.

From Dyerville south only two lines are at all feasible, one following the South Fork and the other following Eel River. The latter was deemed the more desirable. The ruling grades on this line are 0.7 per cent compensated, and the maximum curves are 10 deg. The longest tunnel, at Island Mountain, is about 4300 ft., which saves a detour around a loop of more than 4 miles following the river and avoids some bad sliding country.

Bridges

Three long bridges span the Eel River. Twelve smaller structures just north of Willets, crossing Outlet Creek in a distance of 20 miles, have from two to six steel girders each. Numerous pipe culverts and small concrete arches with ballasted decks span small tributaries.

Topography along Route of Northwestern Pacific Railroad

Of the three steel bridges over Eel River the longest has a length of 1117 ft. and is located at Cain Rock, 10 miles south of Ft. Seward. It consists of three through spans of 200-ft. Pratt trusses on 45-deg. skew piers, with 60 and 70-ft. steel deck girders at the ends. No unusual difficulties were encountered in building the piers under the four central spans, as rock was within 20 ft. or less of the surface. Concrete for the piers is placed from a mounted concrete tower which traverses the entire length of the bridge along a trestle. The higher part of the trestle accommodates a track over which cars carrying the aggregate are brought to a Smith mixer located at the base of the tower. On Oct. 1 pier 6 had been completed, pier 4 had the foundation placed and work had begun on piers 5 and 7. These four large piers are in the river; the small ones out of water were mostly completed or under way at that time.

At Dyerville the crossing is made by two 180-ft. and two 200-ft. spans of through pin-connected Pratt trusses on foundation piers built on piles driven into the gravel and in cribs with cofferdams carried down through sand and gravel to rock from 20 to 25 ft. below low water. The contractor for the foundation, the Mercer-Fraser Company, had an extensive outfit, including A-frame derricks, several pile drivers and a concrete tower.

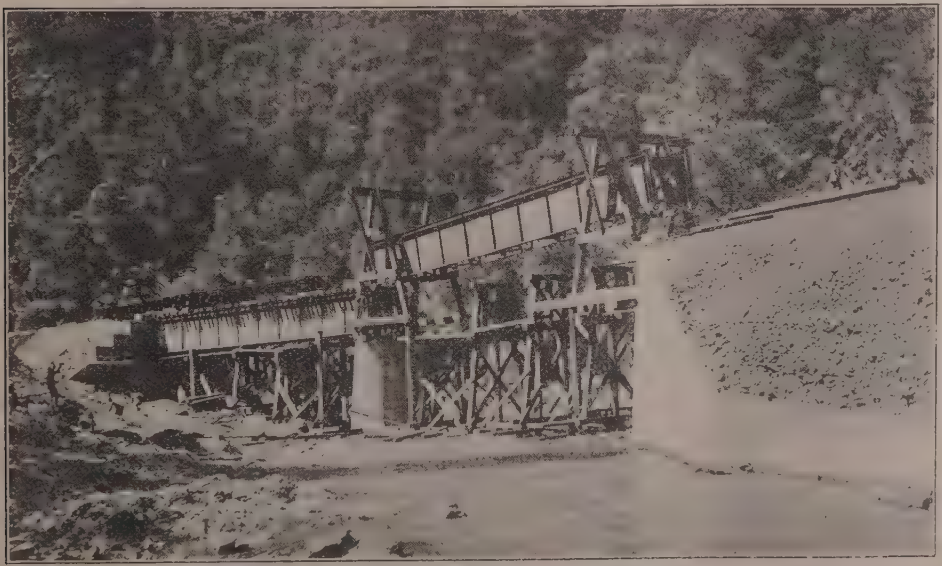
At the base of the latter a ¾-yd. Smith mixer received aggregate from industrial cars hoisted over an incline. Hoppers to receive the mixed concrete from the elevator bucket rested on cantilevers extended in front of the guides.

The third large bridge, 12 miles south of the Cain Rock bridge, spans the river just 20 ft. ahead of the portal of Island Mountain tunnel, about 93 ft. above low-water level. It has two 60-ft. and one 100-ft. deck plate girders and two 200-ft. deck pin trusses.

Tunnels

Between Eureka and San Francisco will be forty-seven tunnels, thirty-one south of Island Mountain and the remainder north. Island Mountain tunnel, directly across the river from the mountain bearing that name, which in turn receives its name from the fact that Eel River, together with two small creeks, almost surround it with water, is a single-track bore driven on a tangent. It enters the steep slope of the mountain at the south portal 20 ft. beyond the end of the 620-ft. river crossing just mentioned.

The tunnel is of the standard Southern Pacific type, 21 ft. 4 in. high by 17 ft. wide, timbered with 10 x 14-in. redwood five-sided bents spaced from 6 in. to 5 ft. apart, depending on the material, which varies from soft shale to sandstone. Isolation from the nearest point of supply, Longvale, 65 miles south, to which point the company constructed a wagon road, made it impracticable to bring in heavy machinery. The faces were therefore hand-drilled and the muck was removed by horses and cars.



Lowering Plate-Girder Span into Place from Gallows Frame

Work on the tunnel was begun in May, 1911, and the two headings came together Feb. 1, 1913, while the benches, which followed the headings within 100 ft., were completely removed by March 17. About 300 men were employed, working three eight-hour shifts per day. The average daily advance during the 631 actual working days was 4 ft. 2 in. Spoil taken from the approaches and tunnel aggregated 93,000 cu. yd. During construction 750,000 lb. of dynamite, 75,000 caps and 300,000 ft. of fuse were employed. About 3,000,000 ft. of lumber were used in timbering, not including 1500 cords of wood packed in the over-break cavities. So well distributed was the work and so evenly did both crews progress that the heading was holed through at the center of the tunnel.

Nels Lind & Company had the contract for the north end and the Utah Construction Company worked from the south portal. The engineer in charge of the work predicted the date of completion within two days, and his lines came together within 1/16 in.

Grading

Much grading work, including the removal of slides and the excavation of gravel ballast, has been carried on by company forces utilizing four 70-ton Bucyrus steam shovels. At the north end several contractors are at work with four shovels, and all contracts have been let for the excavation at the south end. One of the greatest handicaps experienced in the construction of the Northwestern Pacific has been the sliding character of the country all along the line. A cut is seldom made that is not followed by a succession of slides that greatly delay and retard progress. Contractors who succeed in turning over work to the railroad company before a slide has come in to undo all their efforts count themselves fortunate, for then the railway company takes upon itself the task of keeping the right-of-way clear.

Long hauls of supplies and material over the mountain roads are large factors determining the length of time yet required to complete the link. More than 80 miles of wagon road have been constructed, having grades as high as 8 per cent. A large portion of the mileage comprised the connections from the main stage road on the ridge to convenient points on the railroad.

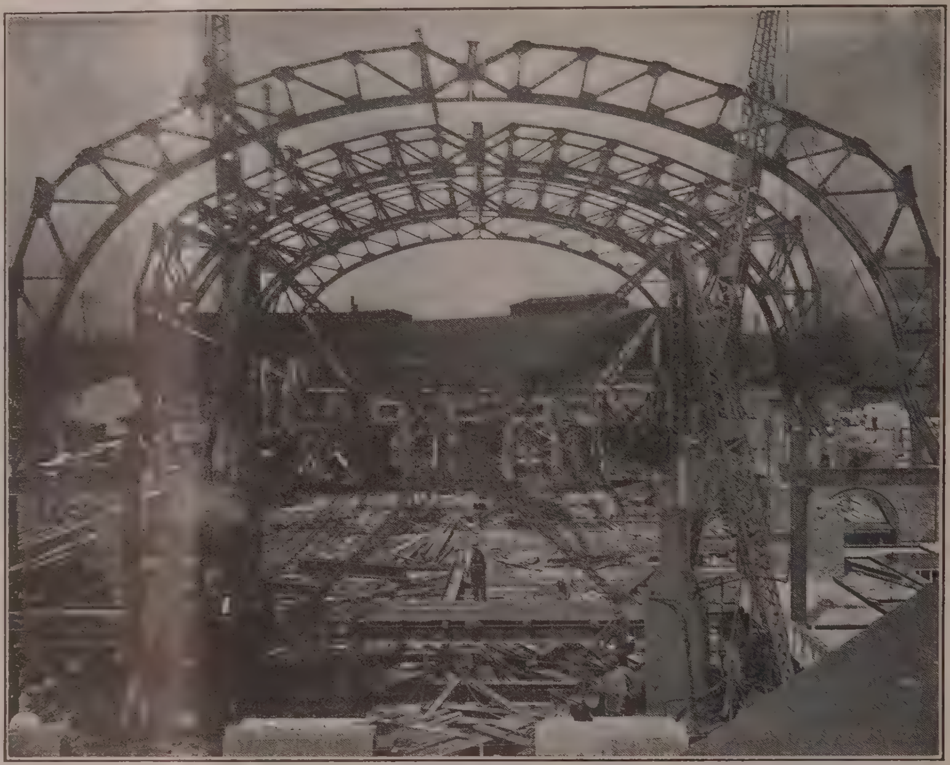
The construction work of the Northwestern Pacific Railroad, which is owned jointly by the Santa Fé and Southern Pacific, is handled by Mr. W. C. Edes, chief engineer. The Utah Construction Company is the general contractor.

Roof Trusses of New Rochester Passenger Station

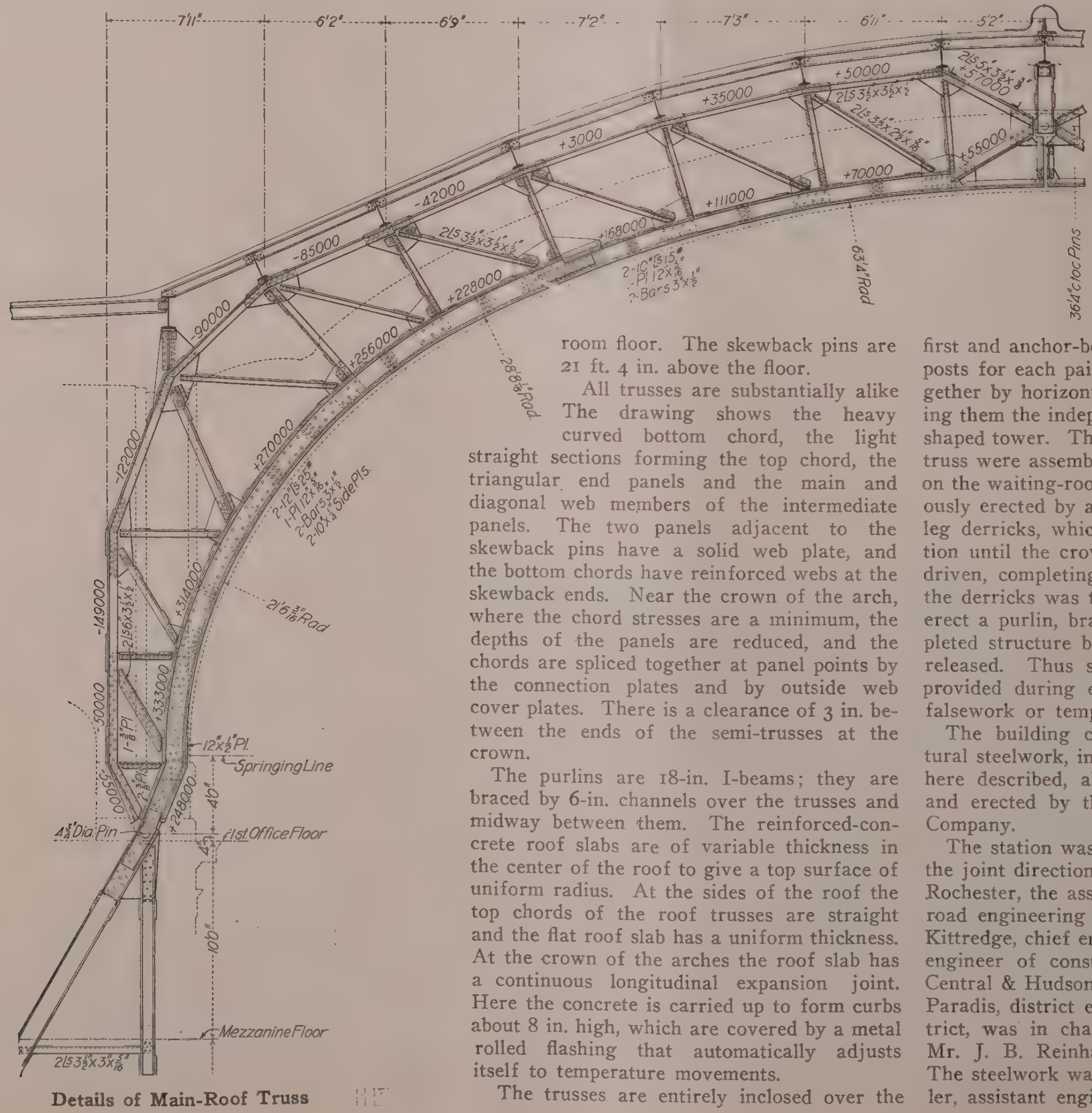
Three-Hinged 90-Foot Steel Arches Arranged in Four Cross-Braced Pairs and Supported on Two-Story A-Frames

THE new passenger station of the New York Central & Hudson River Railroad at Rochester is a large steel frame building occupying a triangular site between Clinton, Central and Joseph Avenues, one block east of the old station. The most important structural feature is the 122 x 156-ft. roof over the main waiting room. This is carried on four pairs of three-hinged steel arch trusses of 90 ft. 8-in. span supported in an unusual manner by long vertical and inclined posts engaging their skewback pins.

There are four pairs of trusses thoroughly braced together with riveted horizontal and transverse braces in the planes of the top and bottom chords and of the web members. The trusses in the end pairs are 9½ ft. on centers and in the intermediate pairs 8 ft. 8 in. The pairs of trusses are uniformly 39 ft. 4 in. apart; they are considered independently stable, and no lateral, diagonal, or sway bracing is provided in the wide panels between the pairs. The trusses have a rise of 36 ft. 4 in. on centers and a chord length of 68 ft. 1 3-16 in. on centers of each semi-truss, and the crown pin is 67 ft. 8 in. above the waiting-room floor. The skewback pins are 21 ft. 4 in. above the floor.



Simultaneous Erection of Semi-Arch Trusses by Steel Traveler



Details of Main-Roof Truss

All trusses are substantially alike The drawing shows the heavy curved bottom chord, the light straight sections forming the top chord, the triangular end panels and the main and diagonal web members of the intermediate panels. The two panels adjacent to the skewback pins have a solid web plate, and the bottom chords have reinforced webs at the skewback ends. Near the crown of the arch, where the chord stresses are a minimum, the depths of the panels are reduced, and the chords are spliced together at panel points by the connection plates and by outside web cover plates. There is a clearance of 3 in. between the ends of the semi-trusses at the crown.

The purlins are 18-in. I-beams; they are braced by 6-in. channels over the trusses and midway between them. The reinforced-concrete roof slabs are of variable thickness in the center of the roof to give a top surface of uniform radius. At the sides of the roof the top chords of the roof trusses are straight and the flat roof slab has a uniform thickness. At the crown of the arches the roof slab has a continuous longitudinal expansion joint. Here the concrete is carried up to form curbs about 8 in. high, which are covered by a metal rolled flashing that automatically adjusts itself to temperature movements.

The trusses are entirely inclosed over the main waiting room and the spaces between them are utilized for a ventilation chamber. The lower chords support the vaulted tile ceiling, which is a Guastavino arch suspended from the trusses.

The roof trusses weigh about 22 tons each, and each semi-truss was shop-riveted in two sections. The vertical and inclined posts supporting each semi-truss were erected first and anchor-bolted to their footings. The posts for each pair of trusses were braced together by horizontal and diagonal struts, giving them the independent stability of a wedge-shaped tower. The two sections of each semi-truss were assembled and riveted together flat on the waiting-room floor and were simultaneously erected by a pair of movable steel stiff-leg derricks, which maintained them in position until the crown and skewback pins were driven, completing the arch section. One of the derricks was then disengaged and used to erect a purlin, bracing it to the already completed structure before the other derrick was released. Thus stability of the trusses was provided during erection without the use of falsework or temporary bracing.

The building contains 910 tons of structural steelwork, including 250 tons in the roof here described, all of which was fabricated and erected by the Buffalo Structural Steel Company.

The station was designed and erected under the joint direction of Mr. Claude Bragdon, of Rochester, the associate architects of the railroad engineering department, and Mr. G. W. Kittredge, chief engineer, and Mr. J. W. Pfau, engineer of construction, of the New York Central & Hudson River Railroad. Mr. F. E. Paradis, district engineer of the Western district, was in charge of all construction and Mr. J. B. Reinhardt was resident engineer. The steelwork was designed by Mr. J. L. Miller, assistant engineer.

Flow of Water in Pipes

Presentation of New Formula Based on Extensive Experiments and Its Application to Wood-Stave, Cast-Iron, Riveted Steel and Concrete Conduits

By E. A. MORITZ,
Engineer, U. S. Reclamation Service, Washington, D. C.

THE so-called "exponential" formula, with a constant coefficient for flow of water in pipes, is gaining an increasingly strong foothold in the confidence of engineers and is destined to replace all other methods of calculating pipe discharge. Although much has been added in recent years to our knowledge of the phenomena of flow of water in pipes, the understanding of these phenomena is still in a very imperfect state.

The matter presented in this article is not intended for the limited number of experienced hydraulicians who have to deal with large and costly works that warrant a complete investigation before final decisions are made. It is for that much larger class of engineers who cannot afford the time and money required for investigating the literature on the subject and gathering experimental data but who want information founded on sound theory and practice.

Experiments and investigations that have been made afford sufficient proof that the loss of head in a pipe—not including losses due to obstructions, curves and bends—is correctly expressed by a formula of the form $H = mV^x/D^z$, where x has a value between 1.7 and 2 and the value of z is very close to 1.25 for all kinds of pipes. It is commonly supposed that x increases with the roughness of the interior surface of the pipe. This may be the general law, but experiments have shown that pipes with apparently similar interior surfaces give widely different values of x and that pipes which are apparently rougher have even given lower values of x than smoother pipes, so that the rational selection of a value of x to fit a given condition of interior surface is practically an impossibility. It will likely be a long time before sufficient data will have accumulated to establish definitely this relation, and for the present it seems more rational to assume a fixed value of x which represents average conditions, as near as may be, and to select different values of m to fit the different kinds of pipe.

WOOD-STAVE PIPE

The writer recently made some extensive and careful experiments (see "Transactions" of the American Society of Civil Engineers, Vol. 74, page 411) on the flow of water in wood-stave pipe from 4 to 55¾ in. in diameter. These tests have since been further verified by additional observations, from which the value of 1.26 was deduced for z, 1.8 for x, and 8.6 for m, when the diameter is expressed in inches, or 0.38 when expressed in feet. The equation in the latter case is $H = 0.38\,V^{1.8}/D^{1.26}$, where H is the friction loss per 1000 ft. of pipe; V, the mean velocity of flow in feet per second; and D, the diameter of pipe in feet. Solving for V, we get

$$V = 1.72\,D^{0.7}\,H^{0.555}$$

and

$$Q = 1.35\,D^{2.7}\,H^{0.555}$$

where Q = discharge in second-feet.

This formula is for wood-stave pipe, either jointed or continuous, in good condition and having ordinarily regular alignment and profile. Additional allowance must be made for loss of head due to sharp bends or curves, valves or other obstructions, entry losses, etc. Two items that in the writer's experience have been most influential in reducing the discharge of pipes are deposits of silt, producing a reduction of cross-sectional area, and air mixed with the water. Unfortunately these two items have been given very little attention by experimenters. The remedy for silt deposits is evident, but the air problem is not so simple. Air-reliefs, located at summits in a pipe, do not give adequate help. The thing to do is to capture the air at the intake and prevent it from entering the pipe. If this is not possible, adequate air-reliefs should be located just below the intake, so as to minimize the amount of air carried through the pipe with the water. It is difficult to emphasize too strongly this air problem as affecting the flow of water in pipes, and it is hoped the matter will receive more attention in the future than it has heretofore. It is not unlikely that this might explain some of the vagaries in experiments which have been made.

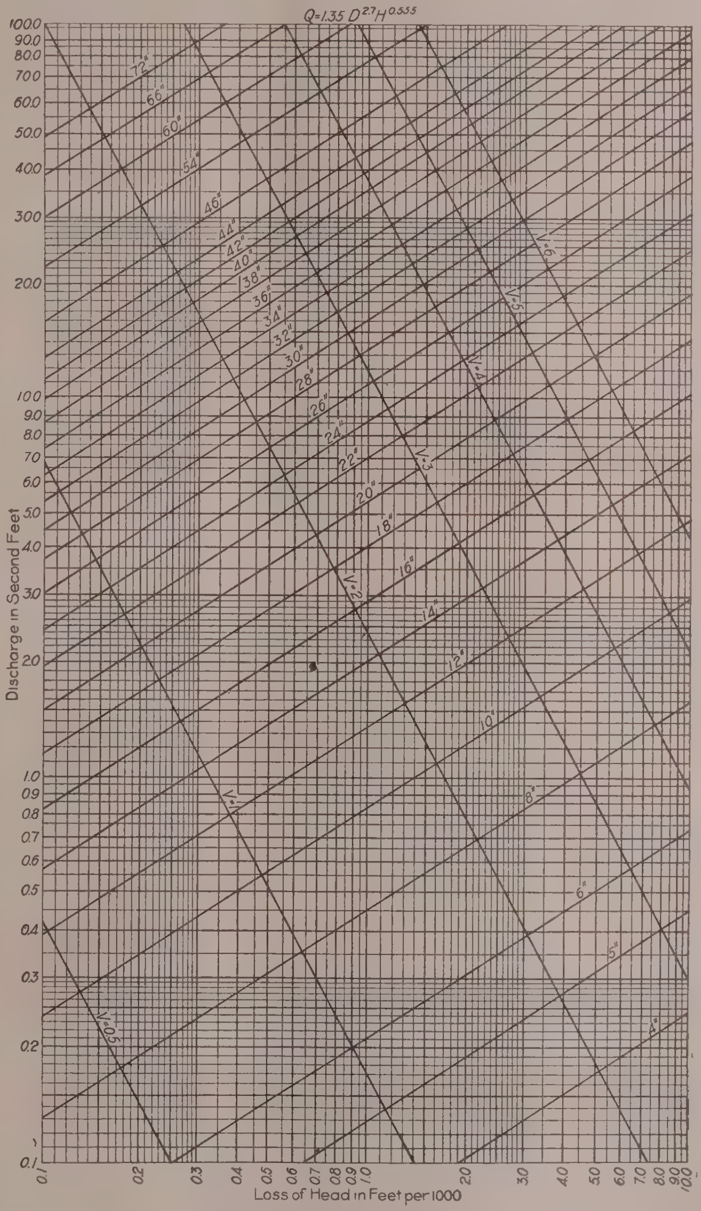


Diagram for the Flow of Water in Wood-Stave Pipes

In Table 1 are given the discharge and velocity for pipes varying from 6 to 120 in. in diameter for a large number of hydraulic slopes. If the discharge and velocity are desired for slopes not shown in the table it will generally be sufficiently accurate to make a linear interpolation between the next lower and next higher slope. If greater accuracy is desired, the quantity may be obtained by calculating the value of $H^{0.555}$ and multiplying by the value of $1.35\,D^{2.7}$ given in the table. The column of values of $1.35\,D^{2.7}$ has been inserted in the table for this purpose. For convenience a column of areas is given and diameters are given in both feet and inches.

The writer has found a logarithmic diagram the most convenient and satisfactory means of representing the flow of water in pipes, and the proposed formula lends itself admirably to a logarithmic plotting. The diagram shown herewith, although covering only limited range, serves to express the idea. All the factors involved in the flow of water in a pipe can be taken from the diagram at a glance. For instance, we read that a 16-in. pipe has an area of 1.4 sq. ft. and that with a velocity of 3 ft. per second it will discharge 4.2 sec.-ft. with a loss of head of 1.9 ft. per 1000-ft. length of pipe. Taking the problem from another angle, suppose we require a capacity of 3 sec.-ft. and there is available a fall of 2 ft. in 1000 ft. From the bottom scale we follow the line for $H = 2$ vertically to its intersection with the horizontal line representing $Q = 3$. This point of intersection lies almost on the 14-in. pipe line, showing that a 14-in. pipe is required; interpolating between the lines for velocity of 2 ft. and velocity of 3 ft., we find the velocity to be 2.8 ft. per second. Thus, in whatever form the problem is presented, the essential factors—diameter, area, velocity, discharge and friction head—are always shown at a glance. This feature is especially serviceable for making preliminary estimates and comparisons.

With the aid of a slide rule, the accompanying table and a sheet of logarithmic paper with appropriate scales a diagram of this kind can be made in a very short time for any kind of pipe and covering any desired range of diameters, velocities, discharges and friction losses. Both the diameter and velocity curves are straight lines; hence only two points need be plotted for each curve. If the logarithmic paper of the desired range

TABLE 1—DISCHARGE IN SECOND-FEET AND VELOCITY IN FEET PER SECOND IN WOOD-STAVE PIPE HAVING ORDINARILY SMOOTH ALIGNMENT AND PROFILE, BASED ON THE FORMULA $Q=1.35\ D^{2.7}\ H^{0.555}$

Diam. *d* in inches	Diam. *D*, in feet	Area in square feet	1.35 $D^{2.7}$	FRICTION LOSS IN FEET PER 1000 FT. LENGTH OF PIPE=*H*																			
				0.2		0.4		0.6		0.8		1.0		1.5		2.0		2.5		3.0		4.0	
				Q	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*
6	0.500	0.196	0.208	0.09	0.46	0.13	0.66	0.16	0.82	0.18	0.92	0.21	1.07	0.26	1.33	0.31	1.58	0.35	1.78	0.38	1.94	0.45	2.30
8	0.667	0.349	0.452	0.18	0.52	0.27	0.77	0.34	0.97	0.40	1.15	0.45	1.29	0.57	1.63	0.66	1.89	0.75	2.15	0.83	2.38	0.98	2.81
10	0.833	0.545	0.824	0.34	0.62	0.50	0.92	0.62	1.14	0.73	1.34	0.82	1.50	1.03	1.89	1.21	2.22	1.37	2.51	1.52	2.79	1.78	3.26
12	1.000	0.785	1.350	0.55	0.70	0.81	1.03	1.02	1.30	1.19	1.52	1.35	1.72	1.69	2.15	1.98	2.52	2.24	2.85	2.48	3.16	2.91	3.71
14	1.167	1.069	2.048	0.84	0.79	1.23	1.15	1.54	1.44	1.81	1.69	2.05	1.92	2.56	2.39	3.01	2.81	3.41	3.19	3.77	3.53	4.42	4.13
16	1.333	1.396	2.933	1.20	0.86	1.76	1.26	2.21	1.58	2.59	1.85	2.93	2.10	3.67	2.63	4.30	3.09	4.88	3.49	5.40	3.87	6.33	4.53
18	1.500	1.767	4.034	1.65	0.93	2.43	1.38	3.04	1.72	3.56	2.01	4.03	2.28	5.05	2.86	5.93	3.35	6.71	3.80	7.42	4.20	8.71	4.93
20	1.667	2.181	5.365	2.20	1.01	3.23	1.48	4.04	1.85	4.74	2.17	5.37	2.46	6.72	3.08	7.88	3.61	8.97	4.09	9.87	4.52	11.6	5.31
22	1.833	2.640	6.932	2.84	1.08	4.17	1.58	5.22	1.98	6.12	2.32	6.93	2.62	8.68	3.29	10.2	3.86	11.5	4.37	12.8	4.83	15.0	5.67
24	2.000	3.142	8.772	3.59	1.14	5.28	1.68	6.61	2.10	7.75	2.47	8.77	2.79	11.0	3.50	12.9	4.10	14.6	4.64	16.1	5.14	18.9	6.02
26	2.167	3.687	10.89	4.46	1.21	6.55	1.78	8.20	2.22	9.62	2.61	10.9	2.95	13.6	3.70	16.0	4.34	18.1	4.91	20.0	5.44	23.5	6.38
28	2.333	4.276	13.30	5.44	1.27	8.00	1.87	10.0	2.34	11.8	2.75	13.3	3.11	16.7	3.90	19.5	4.57	22.1	5.18	24.5	5.72	28.7	6.72
30	2.500	4.909	16.02	6.57	1.34	9.65	1.96	12.1	2.46	14.2	2.89	16.0	3.27	20.1	4.09	23.6	4.80	26.7	5.43	29.5	6.01	34.6	7.06
32	2.667	5.585	19.08	7.81	1.40	11.5	2.05	14.4	2.57	16.9	3.02	19.1	3.42	23.9	4.28	28.0	5.02	31.7	5.68	35.1	6.29	41.2	7.37
34	2.833	6.305	22.46	9.19	1.46	13.5	2.14	16.9	2.68	19.8	3.15	22.5	3.56	28.1	4.46	33.0	5.24	37.4	5.92	41.3	6.55	48.5	7.69
36	3.000	7.069	26.22	10.7	1.52	15.8	2.23	19.8	2.80	23.2	3.28	26.2	3.71	32.8	4.65	38.5	5.45	43.6	6.17	48.2	6.82	56.6	8.00
38	3.167	7.876	30.35	12.4	1.58	18.2	2.32	22.9	2.90	26.8	3.40	30.4	3.85	38.0	4.83	44.6	5.66	50.5	6.41	55.8	7.10	65.5	8.32
40	3.333	8.727	34.83	14.3	1.63	21.0	2.40	26.2	3.01	30.8	3.53	34.8	3.99	43.6	5.00	51.2	5.88	57.9	6.64	64.1	7.35	75.2	8.62
42	3.500	9.621	39.75	16.3	1.69	23.9	2.49	29.9	3.11	35.1	3.65	39.8	4.13	49.8	5.18	58.4	6.07	66.1	6.87	73.1	7.60	85.8	8.92
44	3.667	10.56	45.08	18.4	1.75	27.1	2.57	34.0	3.21	39.8	3.77	45.1	4.27	56.5	5.35	66.2	6.27	75.0	7.10	82.9	7.85	97.3	9.20
46	3.833	11.54	50.80	20.8	1.80	30.6	2.65	38.3	3.31	44.9	3.89	50.8	4.40	63.6	5.51	74.6	6.47	84.5	7.31	93.5	8.10	110	9.51
48	4.000	12.57	57.00	23.3	1.86	34.3	2.73	42.9	3.41	50.4	4.00	57.0	4.53	71.4	5.68	83.7	6.66	94.8	7.54	105	8.34	123	9.79
50	4.167	13.64	63.66	26.1	1.91	38.3	2.80	47.9	3.51	56.2	4.12	63.7	4.67	79.7	5.84	93.5	6.85	106	7.76	117	8.59	137	10.1
52	4.333	14.75	70.74	29.0	1.96	42.5	2.88	53.3	3.61	62.5	4.24	70.7	4.79	88.6	6.00	104	7.05	118	7.98	130	8.83	153	10.4
54	4.500	15.90	78.34	32.1	2.02	47.1	2.96	59.0	3.71	69.2	4.35	78.3	4.92	98.1	6.16	115	7.24	130	8.20	144	9.07	169	10.6
56	4.667	17.10	86.45	35.4	2.07	52.0	3.04	65.1	3.81	76.4	4.46	86.4	5.05	108	6.33	127	7.43	144	8.41	159	9.30	187	10.9
58	4.833	18.35	95.00	38.9	2.12	57.1	3.11	71.5	3.90	83.9	4.57	95.0	5.18	119	6.48	140	7.61	158	8.61	175	9.54	205	11.2
60	5.0	19.64	104.1	42.6	2.17	62.6	3.19	78.4	3.99	91.8	4.68	104	5.30	130	6.64	153	7.79	173	8.82	192	9.76	225	11.4
66	5.5	23.76	134.7	55.1	2.32	81.0	3.41	101	4.27	119	5.01	135	5.67	169	7.10	198	8.33	224	9.34	248	10.4	291	12.2
72	6.0	28.27	170.3	69.7	2.46	102	3.62	128	4.54	150	5.32	170	6.02	213	7.55	250	8.85	283	10.0	313	11.1		
78	6.5	33.18	211.4	86.5	2.61	127	3.83	159	4.80	187	5.63	211	6.37	265	7.98	311	9.36	352	10.6	389	11.7		
84	7.0	38.48	258.3	106	2.75	155	4.04	195	5.06	228	5.93	258	6.71	323	8.41	379	9.86	430	11.2	475	12.4		
90	7.5	44.18	311.2	127	2.88	187	4.24	234	5.30	275	6.22	311	7.04	390	8.82	457	10.4	517	11.7				
96	8.0	50.26	370.4	152	3.02	223	4.43	279	5.55	327	6.51	370	7.37	464	9.23	544	10.8	616	12.2				
102	8.5	56.74	436.3	179	3.15	262	4.63	329	5.79	385	6.80	436	7.69	546	9.63	641	11.3						
108	9.0	63.62	509.0	208	3.28	306	4.81	383	6.03	450	7.07	509	8.00	637	10.0	748	11.8						
114	9.5	70.88	589.2	241	3.40	354	5.00	444	6.26	521	7.34	589	8.31	738	10.4	866	12.2						
120	10.0	78.54	676.6	277	3.53	407	5.18	510	6.49	598	7.61	677	8.62	847	10.8								

Diam. *d* in inches	Diam. *D*, in feet	Area in square feet	1.35 $D^{2.7}$	FRICTION LOSS IN FEET PER 1000 FT. LENGTH OF PIPE=*H*																			
				5.0		6.0		7.0		8.0		9.0		10		15		20		25		30	
				Q	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*	*Q*	*V*
6	0.500	0.196	0.208	0.51	2.60	0.56	2.86	0.61	3.11	0.66	3.37	0.70	3.57	0.75	3.82	0.93	4.74	1.10	5.61	1.24	6.33	1.37	6.99
8	0.667	0.349	0.452	1.10	3.15	1.22	3.50	1.33	3.81	1.43	4.10	1.53	4.39	1.62	4.64	2.03	5.82	2.38	6.82	2.70	7.74	2.98	8.54
10	0.833	0.545	0.824	2.01	3.69	2.23	4.09	2.43	4.46	2.61	4.79	2.79	5.12	2.96	5.43	3.70	6.79	4.35	7.98	4.92	9.08	5.44	9.98
12	1.000	0.785	1.350	3.30	4.20	3.65	4.65	3.98	5.07	4.28	5.45	4.57	5.82	4.85	6.18	6.07	7.73	7.12	9.07	8.06	10.3	8.92	11.4
14	1.167	1.069	2.048	5.00	4.67	5.54	5.18	6.03	5.64	6.49	6.07	6.93	6.48	7.35	6.87	9.21	8.61	10.8	10.1	12.2	11.4	13.5	12.6
16	1.333	1.396	2.933	7.17	5.14	7.93	5.68	8.64	6.19	9.30	6.66	9.93	7.11	10.5	7.54	13.2	9.44	15.5	11.1	17.5	12.5		
18	1.500	1.767	4.034	9.86	5.58	10.9	6.17	11.9	6.72	12.8	7.24	13.7	7.73	14.5	8.20	18.1	10.2	21.3	12.0				
20	1.667	2.181	5.365	13.1	6.01	14.5	6.65	15.8	7.24	17.0	7.80	18.2	8.33	19.3	8.83	24.1	11.0						
22	1.833	2.640	6.932	16.9	6.41	18.7	7.10	20.4	7.73	22.0	8.33	23.5	8.89	24.9	9.42	31.2	11.8						
24	2.000	3.142	8.772	21.4	6.82	23.7	7.55	25.8	8.22	27.8	8.85	29.7	9.45	31.5	10.0	39.4	12.6						
26	2.167	3.687	10.89	26.6	7.22	29.4	7.99	32.1	8.70	34.5	9.36	36.9	10.0	39.1	10.6								
28	2.333	4.276	13.30	32.5	7.60	36.0	8.41	39.2	9.16	42.2	9.87	45.0	10.5	47.7	11.2								
30	2.500	4.909	16.02	39.1	7.98	43.3	8.83	47.2	9.62	50.8	10.4	54.2	11.0	57.5	11.7								
32	2.667	5.585	19.08	46.6	8.35	51.6	9.24	56.2	10.0	60.5	10.8	64.6	11.6	68.5	12.3								
34	2.833	6.305	22.46	54.9	8.71	60.7	9.63	66.1	10.5	71.2	11.3	76.0	12.0										
36	3.000	7.069	26.22	64.1	9.06	70.9	10.0	77.2	10.9	83.2	11.8												
38	3.167	7.876	30.35	75.2	9.42	82.0	10.4	89.4	11.3	96.2	12.2												
40	3.333	8.727	34.83	85.1	9.75	94.2	10.8	103	11.8														
42	3.500	9.621	39.75	97.1	10.1	107	11.2	117	12.2														
44	3.667	10.56	45.08	110	10.4	122	11.5																
46	3.833	11.54	50.80	124	10.8	137	11.9																
48	4.000	12.57	57.00	139	11.1	154	12.2																
50	4.167	13.64	63.66	156	11.4																		
52	4.333	14.75	70.74	173	11.7																		
54	4.500	15.90	78.34	191	12.0																		

cannot be purchased, it can readily be made, with the aid of a table of logarithms, or, better still, the graduations may be transferred from the scales of a common slide rule.

CAST-IRON PIPE

More tests have probably been made on cast-iron pipe than on any other kind, and many formulæ have been proposed as representing the law of flow therein. Some of these formulæ are tabulated in Table 2, which shows the relative discharge obtained by the use of each formula when V is 3 ft. per second. A portion of this table was taken from "Proceedings" of the Institute of Civil Engineers, Vol. 153. For the sake of standardization, the equivalent discharge coefficients in the formula $V = c\sqrt{RS}$ have been calculated for each formula, and the values given in the table represent the relative discharging ability of each pipe per unit of area according to each formula. To illustrate: The Hawksley formula, No. 5, gives the same discharge as the Lampe formula, No. 14, for a 3-in. pipe, but for a 48-in. pipe the ratio of discharges obtained by the two formulæ is 96/136, the Lampe formula having a discharging ability about 42 per cent higher than the Hawksley formula. For any other velocity than 3 ft. per second these figures would be slightly different, the velocity of 3 having been chosen as representing a common value in practice.

By no means are all the formulæ that have been proposed for the flow of water in pipes given in Table 2, but a sufficient number are shown to illustrate the great variety there is to choose from. The relative results obtained by the use of the different formulæ are well illustrated. Many of these have been discarded in modern practice, and it is well known that such formulæ as Eytelwein's, Hawksley's, Neville's, Prony's and Weisbach's are contrary to established facts in that the discharging ability per unit of area is independent of the diameter. The Kutter coefficients as applied to pipes have also been generally discarded.

A considerable degree of agreement is notable in the results from the last six formulæ. The Williams formula is proposed by Gardner S. Williams in the "American Civil Engineers' Pocketbook." An error was apparently made in reducing the formula given in the handbook, and the coefficient does not agree with that in the above table, which was calculated independently. Smith has probably made the most thorough investigation of flow of water in cast-iron pipes, especially in sizes more than 1 ft. in diameter, and his coefficients are generally accorded considerable weight. The use of these coefficients, however, is a cumbersome process, and the writer proposes the formula $Q = 1.31\ D^{2.7}\ H^{0.555}$, which entirely avoids the use of a table of coefficients and gives results practically identical with Smith's coefficients. To illustrate the relation between the results obtained by the two methods, Table 3 has been prepared. It will be noted that the maximum variation from Smith's coefficients, shown in the table, is about 4 per cent for a 1-ft. pipe and a velocity of 10 ft.,

TABLE 2—DISCHARGE COEFFICIENTS c FOR VARIOUS FORMULÆ REDUCED TO THE FORM $V=c\sqrt{RS}$, WHEN $V=3$ FT. PER SECOND

	Author	DIAMETER OF PIPE									Formulæ
		3 in.	6 in.	12 in.	18 in.	24 in.	30 in.	36 in.	42 in.	48 in.	
1	Bazin	92.2	110.7	125.3	131.7	135.2	137.5	139.1	140.3	141.1	$V=\left(\dfrac{RS}{0.0000457\left(1+\dfrac{0.0984}{R}\right)}\right)^{1/2}$
2	Crimp and Bruges	78.1	87.7	98.4	105.3	110.5	114.7	118.2	121.3	124.0	$V=1.24\, R^{2/3}\, S^{1/2}$
3	Darcy	98.5	105.3	109.3	110.7	111.5	111.9	112.2	112.5	112.6	$V=\left(\dfrac{2gRS}{\zeta}\right)^{1/2}$; $\zeta=\left(0.004346+\dfrac{0.0003992}{D}\right)+\left(\dfrac{0.0010182+\dfrac{0.000005205}{D^2}}{V}\right)$
4	Eytelwein	103.8	103.8	103.8	103.8	103.8	103.8	103.8	103.8	103.8	$V=(11704\, RS+0.01698)^{1/2}-0.1303$
5	Hawksley	96.0	96.0	96.0	96.0	96.0	96.0	96.0	96.0	96.0	$V=48\,(DS)^{1/2}$
6	Kutter	57.8	69.6	85.3	94.4	101.1	106.0	110.1	113.4	116.5	$V=\left(\dfrac{41.6+\dfrac{1.811}{n}+\dfrac{0.00281}{S}}{1+\left(41.6+\dfrac{0.00281}{S}\right)\dfrac{n}{\sqrt{R}}}\right)\sqrt{RS}$ $n=0.013$
7	Lawford	70.0	86.0	105.5	117.5	123.0	127.0	130.8	134.5	138.3	$V=c\sqrt{RS}$. c obtained from experiments.
8	Neville	104.1	104.1	104.1	104.1	104.1	104.1	104.1	104.1	104.1	$V=140\sqrt{RS}-11\sqrt[3]{RS}$
9	Prony	94.5	94.5	94.5	94.5	94.5	94.5	94.5	94.5	94.5	$V=(9419.7\, RS+0.00665)^{1/2}-0.0816$
10	Unwin	100.1	104.6	109.4	112.2	114.3	115.9	117.3	118.4	119.4	$V=\left(\dfrac{2gSD^x}{M}\right)^{1/n}$ $M=0.0215$ to 0.0440 $n=1.72$ to 2.00 $x=1.10$ to 1.39
11	Weisbach	103.0	103.0	103.0	103.0	103.0	103.0	103.0	103.0	103.0	$V=\left[\left(\dfrac{1}{0.01439+\dfrac{0.017155}{\sqrt{V}}}\right)2gDS\right]^{1/2}$
12	Smith	.....	.....	109	116	121	125	128	131	134	$V=c\sqrt{RS}$ c obtained from experiments.
13	Fanning	98	102	106	111	116	.....	123	.....	131	$V=\left(\dfrac{2gDS}{4f}\right)^{1/2}$ f obtained from experiments.
14	Lampe	96	104	114	121	125	129	132	134	136	$V=77.68\, D^{0.694}\, S^{0.555}$
15	Flamant	99	106	113	118	122	125	126	128	129	$V=(76.28$ to $86.38)\, D^{5/7}\, S^{4/7}$ old pipes new
16	Williams	93	101	110	116	120	123	126	129	131	$V=67.7\, D^{0.668}\, S^{0.535}$
17	Moritz	93	102	112	119	122	125	128	130	132	$V=77\, D^{0.7}\, S^{0.556}$

and for the usual diameters and velocities met with in practice the difference is less than 2 per cent, a negligible quantity from a practical standpoint.

This formula differs from that for wood-stave pipe only in the constant coefficient, this for the former being 1.35 and for the latter 1.31. The velocity and discharge for cast-iron pipe may, therefore, be obtained from the table for wood pipe by multiplying the figures therein by 1.31/1.35 = 0.97. This formula is for new cast-iron pipe with smooth alignment and profile. Additional allowance must be made for other than friction losses.

It is generally desirable to make allowances for the deterioration of cast-iron pipe with age. For this purpose we may assume that the friction head increases 3 per cent per year due to tuberculation, and that the diameter decreases 0.01 in. per year from the same cause. These assumptions are made in Williams and Hazen's "Hydraulic Tables." Applying these factors to the equation $Q = 1.31\, D^{2.7} H^{0.555}$, and letting K equal ratio of discharge at age of N years to discharge new, we get $K = [1 - N/1200\, D]^{2.7} \times [1/(1 + 0.03\, N)]^{0.555}$. Thus from this equation we learn that a 12-in. pipe 10 years old will carry 85 per cent as much as the same pipe new, and at the age of 100 years it will carry only 36 per cent.

RIVETED STEEL PIPE

Experiments on riveted steel pipes are too few and inconsistent to give an indication of the law of flow. Such experiments as have been made were on pipes having very different conditions of interior surface, and the results obtained have been very discordant. There is no good reason for assuming that the law of flow is any different than for cast-iron pipe except that the friction losses generally run somewhat higher and the discharges correspondingly lower. The writer has adopted the formula $Q = 1.18\, D^{2.7} H^{0.555}$, which gives results about 10 per cent smaller than the formula for cast-iron pipe. This agrees as well with experiments as can be expected from any formula. It is intended to be used only for new asphalted pipe having a smooth alignment and profile. If it is intended to allow for deterioration with age, and no better information is available, the formula for decrease in carrying capacity given above may be used. The discharges and velocities for riveted steel pipes may be obtained from Table 1 by multiplying the figures therein by 1.18/1.35 = 0.875.

Concrete pipe is manufactured in various ways and by different methods. We have the dry-mix pipe, built in short sections and laid and jointed in a similar manner to clay sewer pipe, and the wet-mix pipe, either built and laid in short sections as aforementioned or built continuously in the trench. In the former case there is bound to be more or less roughness at the joints, with possible jogs in the alignment, while in the latter the continuity is unbroken and better alignment may be obtained. The discharging ability of the continuous pipe is probably as high as that of wood-stave pipe, while the wet-mix jointed pipe may better be classed with cast-iron pipe. The dry-mix pipe has a considerably rougher surface than the wet mix, and on account of the short sections (usually 2 ft.) in which it is built its discharging ability is probably not greater than that of riveted steel pipe, even if it is carefully laid and jointed; if it is not properly laid its carrying capacity may be considerably less.

The writer has knowledge of only one set

TABLE 3—COMPARISON OF FORMULA $Q=1.31\, D^{2.7}\, H^{0.555}$ WITH SMITH'S COEFFICIENTS FOR NEW CAST-IRON PIPE IN THE FORMULA $Q=Ac\sqrt{RS}$

Diam. in ft.	VELOCITY IN FEET PER SECOND														
	2			3			4			5			10		
	Smith	Moritz	Ratio	Smith	Moritz	Ratio	Smith	Moritz	Ratio	Smith	Moritz	Ratio	Smith	Moritz	Ratio
1.0	104	107.5	1.029	109	111.8	1.026	112	115.0	1.027	114	118.3	1.038	121	126.0	1.041
1.5	111	113.5	1.022	116	118.7	1.023	119	122.7	1.031	121	124.7	1.030	129	132.7	1.028
2.0	116	116.8	1.007	121	121.8	1.007	124	124.3	1.002	127	127.3	1.002	135	138.0	1.022
2.5	120	120.8	1.007	125	125.3	1.002	128	127.4	0.996	131	130.8	0.999	137	141.8	1.035
3.0	124	123.0	0.992	128	127.7	0.998	132	130.7	0.991	134	135.2	1.009	143	145.3	1.016
3.5	127	126.5	0.996	131	130.3	0.995	135	132.9	0.985	137	138.0	1.016	146	148.5	1.017
4.0	130	128.6	0.990	134	132.3	0.988	137	136.8	0.999	140	140.4	1.002	150	152.0	1.013
5.0	134	132.3	0.988	139	137.8	0.992	142	141.9	1.000	145	144.9	1.000	155	155.0	1.000
6.0	138	135.3	0.981	142	141.0	0.993	146	145.0	0.994	148	148.5	1.003	....	.....	.....
7.0	141	138.3	0.980	145	143.8	0.992	148	148.0	1.000	151	151.3	1.002	....	.....	.....
8.0	143	140.5	0.983	148	146.4	0.990	151	150.6	0.998	153	154.1	1.007	....	.....	.....

of experiments made on the flow in concrete pipe. These were made on pipes of 46, 30 and 16 in. diameter, the 46 and 30-in. pipes being jointed and manufactured with a wet mix, while the 16-in. pipes were jointed pipes manufactured with a dry mix. The former may, therefore, be classed with cast-iron pipe and the latter with riveted steel pipe.

Table 4 shows how the experimental results compare with the results obtained from the formulæ. The difference in the 46 and 30-in. pipes may readily be attributed to experimental errors and the presence of silt deposits. The methods used in making the experiments were such that an error of 5 per cent in the results of a single experiment may readily have occurred. The 16-in. pipes show a large disagreement between experimental and calculated values, but the disagreement is hardly greater than that shown in the individual experiments themselves, which points strongly to some abnormal condition in the pipes. For example, the discharge of the first 16-in. pipe is very little greater than that of the third pipe, which has only half the fall. The most probable explanation is that large deposits of silt have lodged in the pipes, due to the comparatively low velocities. It is, of course, impossible to provide for this condition in the design.

Table 4—Comparison of Formulæ with Experiments on Concrete Pipes

Diam. in inches	Slope	Velocity in feet per second	Discharge, experimental, in second-feet	Discharge $Q=1.31 D^{2.7} H^{0.555}$ in second-feet	Ratio
46	0.00087	4.00	46.15	45.5	0.986
46	0.00078	3.98	45.90	42.9	0.935
46	0.00108	4.17	48.16	51.2	1.053
46	0.00102	4.21	48.59	49.9	1.027
				Average	1.000
30	0.00107	3.61	17.70	16.2	0.916
30	0.00105	3.39	16.62	16.0	0.963
30	0.00105	3.47	17.04	16.0	0.939
30	0.00086	2.73	13.38	14.6	1.090
				Average	0.977
				$Q=1.18 D^{2.7} H^{0.555}$	
16	0.0038	2.68	3.74	5.84	1.561
16	0.0045	3.49	4.87	6.42	1.318
16	0.0019	2.45	3.42	3.92	1.145
				Average	1.341

Mains to Annexed Districts of Los Angeles

The conditions under which outlying sections may be annexed to Los Angeles and supplied with Owens River water were adopted by the Municipal Annexation Commission Nov. 7. No section may be supplied with water unless it be annexed, when it shall bear its share of the aqueduct, power and harbor bonds, but not of the remainder of the city's indebtedness. Remote annexed districts may have local public-works boards to act under the direction of the central board. Agricultural sections will be remitted 60 per cent of their general taxes to constitute a road and improvement fund; to offset this privilege such aids as parks, playgrounds, garbage collections and, to a large extent, expenses for fire and police protection will not be afforded. The light and power system is to be extended to the new sections, as is connection to the new outfall sewer to the sea.

The First Pile for the foundations of the educational buildings of the new Technology at Cambridge, Mass., was driven on Dec. 4 and the first shipment of steel reinforcement was received Dec. 5.

Drilling as a Test for Concrete

Log of Experiments to Determine Relation between Compressive Strength and Speed of Boring in Concrete

By C. S. DUKE, C.E.
Madison, Wis.

The investigation described in this article was undertaken in order to determine the relation between the compressive strength and speed of drilling of concrete when the personal element is eliminated. It was carried on as thesis work at the University of Wisconsin and was prompted by a controversy which arose over the quality of concrete in the Southwest Land and Lake Tunnel, Chicago. In order to determine the quality, holes were drilled by hand in the concrete lining. The writer disagreed with the conclusions and, therefore, undertook these tests. The record of a portion of the hand-drilling tests is shown in Fig. 3.

The compressive tests were performed in accordance with the rules at the testing laboratory of the University of Wisconsin. The specimens upon which the compression tests were made were cylinders about 6 in. in diameter and 18 in. long. These specimens were made from the same materials and in the same manner as in the case of the boring specimens. All specimens were broken in a 100,000-lb. Riehle testing machine.

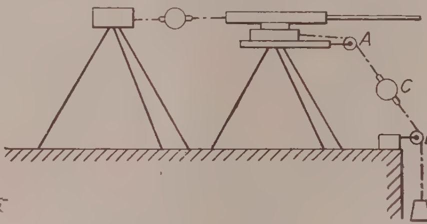


Fig. 1—Arrangement for Constant Feed Pressure

Specifications for Drill Tests

Object.—The purpose of this experiment is to determine the speed of boring through concrete by mechanical means, when the personal element is eliminated. The ultimate object is to discover the relationship, if any, between the compressive strength of different kinds of concrete and the speed with which it can be bored. This line of experimentation is warranted, first, because of the fact that concrete tunnel linings are sometimes examined both as to the quality of the concrete and the depth of the lining by boring, and, second, because, as concrete is frequently used both in temporary and permanent building construction, it becomes necessary at times to wreck such structures, and in so doing boring is resorted to.

Apparatus.—In making complete tests two sets of apparatus shall be used. The first set is to be used in the compression tests and the second in the boring tests. The compression tests are to be performed in accordance with the standard specifications in use at the mechanical laboratory of the University of Wisconsin, and will not be discussed here. The boring tests will be made with the use of the Fort Wayne Electric Company's rock drill, type A, from which the ordinary hand feed is removed and an automatic feed with a weight substituted. There shall be suitable apparatus or means for maintaining the specimen in a rigidly fixed position with respect to the drill. Drill steels shall be of 1 7/16-in. diameter and sharp, as are usually employed in ordinary drill practice. No water shall be poured into drill hole to accelerate the boring speed.

Method of Test.—The drill shall be set up firmly, and thoroughly inspected to insure its good and efficient working order before any specimens are tested. The feed pressure in all the tests shall be about 70 lb. During the boring each drill shall be inspected from time to time, and it shall be replaced by a sharper one when it begins to grow dull. The boring shall proceed through periods of time as conditions shall dictate, and at approximately 860 r.p.m.

Data.—The data required will be the depth of each hole, the time, the total number of revolutions, the materials of which the concrete is made, their proportion, as well as the unit stress in compression for each specimen.

General.—In standardizing the methods of boring in these experiments conditions as near as those met with in actual rock drill work as possible are sought after, so that the result may be applicable to the ordinary cases of rock or concrete drilling as conducted at the present time.

Test Pieces

There were two kinds of specimens used. The first had been used in permeability tests made at the university. They had the general shape of a truncated cone, 20 in. in diam-

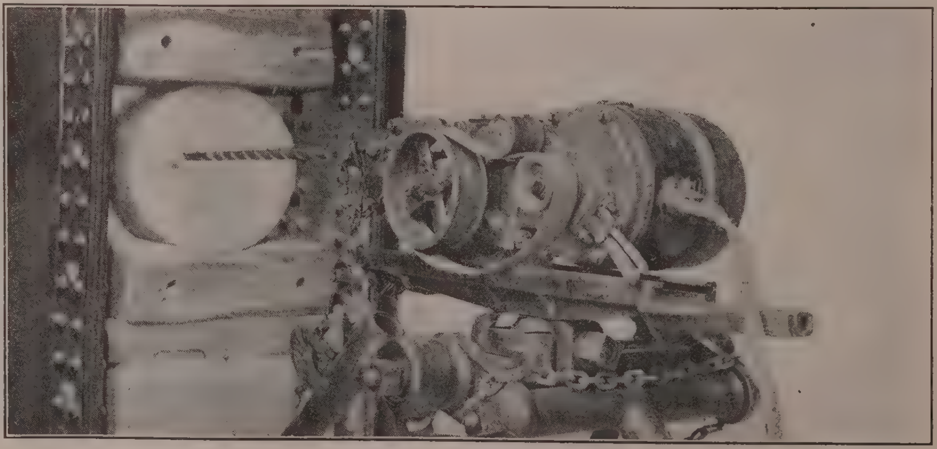


Fig. 2—Boring Apparatus Showing Method of Securing Test Specimen

eter at the base and 18 in. at the top. The height was 9 in. For the purpose of conducting the permeability tests, bottomless cast-iron jars, 16 in. in diameter, had been embedded in the concrete.

The other specimens were also in the shape of truncated cones with 20 and 18-in. diameter end areas. There were three of these, two 8 in. and one 6 in. high. The stone used was Niagara limestone taken from the marine section of the Southwest Land and Lake Tunnel in Chicago.

Drill and Motor

An electric rock drill manufactured by the Fort Wayne Company, of Madison, Wis., was used. It is known as type A, form 1, size 10, serial number 154. The motor for driving the drill was a d.c., 115-volt, 21-amp, 4-pole, compound-wound, 25-hp machine of 2200 r.p.m. It is known as type D, form C, serial number 32,455. The feed screw was removed from the drill so that it could slide freely in the guides, which were oiled so as to reduce the friction as much as possible. The description will be made clearer by reference to Figs. 1 and 2.

The feed pressure was supplied by a system of pulleys and weights. Referring to Fig. 2, *A* and *B* represent the pulleys, *C* the spring balance, and *D* represents the weight. The drill was mounted on an extra-heavy tripod. The points of the legs of the tripod were fixed in small depressions made in the concrete floor. A chain was passed over the upper part and fastened by a turnbuckle to an anchor concreted into the floor. Before fastening the tripod its upper surface was put in a horizontal plane by the use of a spirit level.

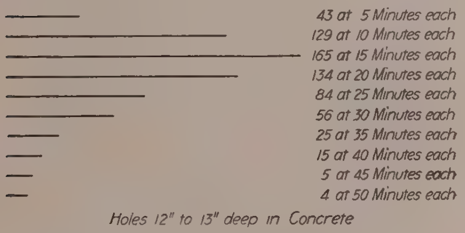


Fig. 3—Record of Hand-Drilled Holes

The specimen was held firmly in the jaws of a large hydraulic testing machine. In order to secure a firm bedding, 6-in. timbers were hollowed out to the curvature of the specimen, and with other blocking placed in between the jaws of the machine and the specimen.

On an average one sharp drill was used for each new specimen. The motor was served with 115 volts and the current was recorded by an ammeter.

Feed Pressure

In order to determine the net feed pressure transmitted to the specimen, another tripod (Fig. 1) was set 4½ ft. to the rear of the machine and in line with it and the specimen. Another spring balance was fastened between the drill and the second tripod by a thin wire rope. This rope was placed as near as possible in the same horizontal line as that described by the center of gravity of the drill as it moved in its guides. The machine was put in operation, and after a number of trials the two spring balances were found to read practically the same.

Normally, the drill makes 860 r.p.m. This was tested and it was found that the speed was not quite constant. It was therefore decided to take the total number of revolutions for the drilling of each hole. The time was taken by a stop watch and the revolutions from the shaft of the revolving helves. The drill was struck two blows, one by each hammer for each revolution.

Drilling

Before beginning the drilling of each hole a slight depression was made so that, when starting, the drill would be held in one place. Preparatory to drilling the machine was moved back until the steel was about 4 in. from the concrete. The drill was then started revolving and allowed to approach the concrete at a moderate speed. When the drill touched the concrete an assistant gave the signal, and from that instant time and revolutions were recorded. Two marks had been placed on the ways of the machine, giving the depth of the hole to be drilled. When the second mark was reached the signal was given and the counting stopped.

Weights amounting to 70 lb. were used in all but two instances. For the 70-lb. weight the net feed pressure transmitted was 68 lb. The no-load current was 2.5 amp, and the full-load varied from 14 to 16 amp. The table gives the results of the tests.

Boring Data Giving the Results of Tests

Total revolutions	Total time, sec.	R.p.m.	Depth drilled, in.	Excavation per min., cu. in.	Excavation per blow, cu. in.	Remarks
348	24.8	845	3.25	12.80	0.00758	Parts by volume, 1:1.3.7:3.30; parts by weight, 1:1.25:3.75; age, 118 days; compression, 2195 lb. per sq. in.; age of compression specimen, 30 days.
409	28.8	853	3.31	11.20	0.00656	
513	36.8	838	4.12	10.90	0.00650	
476	32.8	872	3.94	11.70	0.00672	
420	26.4	960	3.50	11.80	0.00615	Parts by volume, 1:2.48:2.40; parts by weight, 1:2.25:2.75; age, 104 days; compression, 2895 lb. per sq. in.; age of compression specimen, 30 days.
420	20.8	887	3.50	15.00	0.00845	
320	...	...	2.87	...	...	
...	19.8	...	...	...	...	
376	26.2	860	4.37	14.90	0.00865	
315	21.5	870	3.44	14.25	0.00820	
325	22.5	867	3.06	12.15	0.00700	
295	20.6	860	3.06	8.94	0.00518	
...	22.5	...	5.00	23.60	...	Parts by volume, 1:3.30:5.27; parts by weight, 1:3:6; age, 88 days; *age, 89 days; compression, 1220 lb. per sq. in.; age of compression specimen, 60 days.
...	14.0	...	...	...	...	
...	18.8	...	3.75	21.10	...	
248	18.0	832	3.81	18.85	*0.01145	
264	16.8	945	3.81	20.20	*0.01740	
285	21.2	806	4.37	18.40	*0.01141	
247	17.2	855	4.19	14.60	*0.00854	
251	17.4	865	3.87	13.40	0.00775	Parts by volume, 1:3.30:5.27; parts by weight, 1:3:6; age, 90 days; compression, 1220 lb. per sq. in.; age of compression specimen, 60 days.
281	21.4	788	3.50	9.81	0.00623	
244	17.2	853	3.25	16.80	0.00984	
...	...	...	6.00	...	...	
268	16.6	970	3.00	16.10	0.00830	
296	18.5	960	3.87	18.70	0.00974	
315	21.4	884	4.00	11.20	0.00635	
379	25.4	895	3.81	13.40	0.00750	
486	33.2	880	4.18	12.30	‡0.00698	Parts by volume, 1:1.5:3 gravel; parts by weight, 1:1.65:3.42 gravel; age, 84 days; compression, 2700 lb. per sq. in.; age of compression specimen, 60 days.
528	35.8	887	4.18	11.40	‡0.00642	
564	40.2	892	4.62	11.20	‡0.00628	
552	37.8	880	4.18	10.80	‡0.00614	
630	44.0	860	4.75	11.50	0.00674	Parts by volume, 1:1.5:3 gravel; parts by weight, 1:1.65:3.42 gravel; age 84 days; compression, 2700 lb. per sq. in.; age of compression specimen, 60 days.
525	36.0	875	4.50	13.20	0.00753	
...	39.0	...	5.17	12.50	...	
515	34.6	895	4.12	11.70	0.00654	
570	40.0	855	4.12	10.00	0.00585	
622	42.2	880	4.53	9.75	0.00553	
726	50.0	872	4.62	9.00	0.00516	
751	43.0	850	4.19	9.50	0.00451	
320	21.6	890	2.31	10.40	0.00585	Parts by volume, 1:0.91, mortar; parts by weight, 1:1, mortar; age, 62 days; 1:2 mortar showed 5100 lb. per sq. in. after 60 days.
250	17.4	860	1.81	10.10	0.00587	
312	20.6	910	2.06	9.75	0.00535	
248	16.8	886	1.81	10.50	0.00593	
1396	98.6	850	14.81	14.65	†0.00863	Parts by volume, 1:3:6; parts by weight, 1:2.56:6.60; age, 49 days; compression, 778 lb. per sq. in.; age of compression specimen, 52 days.
1542	106.6	865	13.81	12.70	†0.00733	
1635	111.2	860	14.12	12.40	0.00720	
...	79.8	...	10.31	12.60	...	
1542	106.2	870	15.18	13.90	0.00798	
1070	73.8	870	11.12	14.70	0.00844	

The net feed pressure was 68 lb., except for †, where it was 51 lb. Janesville sand, Janesville gravel and Madison stone were used, except for the six last tests, where Niagara limestone was used. Universal Portland cement was used for all tests. All drills used were 1 7/16 in., except in those places where ‡ occurs, where 1⅜ in. drills were used.

Conclusions

As a result of the tests the following conclusions were reached:

1. The experiments were not extended enough to make it clear whether there is any definite law existing between the boring speed and compressive strength of all grades of concrete.

2. The supply of electrical power to the motor was at times not constant, and there were also occasional slight vibrations of the specimen.

3. There was undoubtedly some difference in the sharpness of even new drills.

4. Each drill grew duller as the boring progressed. The rate at which different drills grew duller varied.

5. Despite these facts, however, it would appear that, all things being equal, the rate of boring concrete would, in a general way, vary inversely as the compressive strength.

In view of the wide variation in results it is believed that the conclusions drawn from Fig. 3 are somewhat misleading. The report in which these conclusions were drawn states: "After the drillers had completed the holes in Drift 20 they had become quite expert and their work was fairly uniform, so that it was possible to form a pretty good idea of the character of the concrete from the time it took to drill the holes to a depth of 12 in., and the men were average drillers." That the drilling furnished a very unreliable criterion as to the character of the concrete is evident for the following reasons: (1) The variation in the age of the concrete, it being from two and one-half years to six weeks old; (2) the personal equation of the drillers; (3) the condition of the drill as to sharpness or dullness; (4) the failure to differentiate between these factors.

Road Improvement in British Columbia made during the past season, according to a report from the Minister of Public Works, cost a total of $4,000,000, of which $900,000 was for bridges, $100,000 for road-working machinery and $70,000 for road-location work.

American Road Builders' Association

Abstracts of Some of the Papers Presented at the Annual Convention in Philadelphia, December 9-12

Sub-Organization for Maintenance

By **JOHN N. CARLISLE**

Commissioner of Highways, State of New York

MAINTENANCE is the really great problem confronting those who are in charge of State highway departments. It is a problem which involves, first, the question of what scheme of organizing a department should be followed.

Formerly maintenance work in New York was separate from construction, and was in charge of a maintenance bureau under different divisions and under superintendents of maintenance. This plan of divided authority was so unsatisfactory that in the winter of 1913 the State Legislature provided that both departments should be consolidated under the control of the division engineers, and that the work of maintenance should be done either by contract or directly by the department.

New York State is now divided into nine divisions, with a division engineer in charge, and we intend to subdivide each of these divisions into about seven subdivisions. In charge of both construction and maintenance in each of these subdivisions we intend to appoint an assistant engineer, who will be held responsible for the condition of his roads.

The New York plan in the past has been to rely largely upon the patrol system—placing a man with a horse and wagon in charge of a short number of miles and imposing upon him the duty of keeping his section of the road in repair. While we do not believe in entirely abandoning this system, we are arranging for an organization of section gangs in each of the subdivisions. Each section gang is to be furnished with equipment with which roads can be oiled and repaired.

We believe that in following the methods of the steam railroads in taking care of their tracks we can accomplish better results than in any other way. This problem, of course, means the organization of a proper section gang with equipment sufficient to do repair work. Our plans call for the furnishing of an auto truck, pressure sprayer oiler, steam roller, and heating tanks, together with small tools. It is believed that each section gang will be large enough and well enough equipped to clean and oil the roads and make temporary and permanent repairs on any type of a road already constructed.

Realignment, Grading and Drainage

By **S. D. FOSTER**

Chief Engineer, Pennsylvania State Highway Department

DETERMINATION of the amount of grading to be done in connection with the improvement of any highway depends upon the amount and nature of the travel and the topography. Excessive grades increase the cost of transportation and also add a heavy burden of maintenance, and it is always advantageous to lengthen a road and eliminate an excessive grade. This, as a whole, should be governed by the following axioms:

1. No greater load can be moved over a highway than is moved over the maximum grade. 2. It is generally true that a road over a hill is equal in length to one around the base. 2. If the tendency of the topography is toward a continued elevation, the grade line should never be allowed to have a descending grade, and vice versa.

Sacrifice of straightness or of grade will be dependent upon the predominant nature of the travel. Where horse-drawn vehicles predominate, alignment should give way to the lessening of the grade, as horse-drawn vehicles demand easy grade in preference to straight roads; where motor travel predominates, the grades should give way to alignment, for the rapidly increasing use of motor vehicles places a new responsibility upon the roadmaker—the reduction of danger to life and property, which necessitates building highways with long easy curves.

Drainage

In determining the amount of drainage it is well to remember that the ability of earth or soils to sustain loads depends largely upon the moisture present. Most soils can be compacted to form a good, firm foundation as long as they are kept dry, but when wet they become soft and in a great measure lose their sustaining power. The main problem, therefore, in the construction of a highway is drainage, which is dependent upon proper location. If the ground is level and not subject to flood there is seldom difficulty in location, but marshy ground underlaid with quicksands is expensive to deal with and generally should be avoided, even though it involves a considerable detour. A quicksand well drained often makes an excellent foundation for a road; the drainage, however, must be thorough and rapid. If the ground is level and subject to flood, mainly by backwater, an adequate number of small drains will usually accomplish the desired result, except that it may cost a good deal to raise the grade of the road above flood level. Cases sometimes arise where it is more economical to build a road floodproof at a lower level and submit to an occasional interruption of traffic than to make a wide detour or go to the great expense of a fill or viaduct.

Roads Subjected to Floods

If the ground is level and subject to floods running at high velocity great care is necessary. The natural channel of the stream is altogether insufficient to carry the flood water and, if the bridge crossing the stream is made large enough to permit the flow to pass under it, extensive erosion is liable to take place. If overflow bridges are built they are very likely to cause formation of side channels. Usually the best treatment is to build the bridge large enough to pass the whole stream in flood time, straighten and widen the channel of the stream as much as possible, remove obstructions and protect the banks at exposed points.

In some cases it may be necessary to allow the flood water to overflow the road at some points, which must, of course, be made floodproof. Where such overflow points are provided, they should be made long and shallow to reduce as much as possible the velocity of the current flowing over them; otherwise adjacent property will suffer unnecessarily. These low places should also be so located as to reduce the actual damage to the minimum, as it is the duty of the public official to protect the just rights of all parties. If the proposed road crosses a narrow valley with a rapid stream, it is almost always best to give the bridge ample waterway and fix the grade above the reach of high water, taking care, of course, to protect all exposed points against erosion. Where valleys are to be crossed it is always better to cross them where they are narrow and the streams rapid. Such conditions reduce the amount of embankment required and also the span of the bridge, requiring more substantial construction, but proving more economical and more enduring.

One other condition met with frequently upon the highways of Pennsylvania is that arising from the tendency of one stratum of soil to slip upon another and thus cause land slides. These are most difficult and expensive to deal with, and it is seldom if ever worth while to attempt to hold such a movement of earth in place by piles or retaining walls. The only sure treatment is the thorough cutting off of the water before it enters the sliding mass. French drains with suitable laterals are generally better than tile drains, the usefulness of which may be destroyed by a slight movement in the slide. The drainage must be ample, as regards the sizes and lengths of the drains and the sufficiency of the outlets. This is sometimes a very expensive treatment, but if well done a permanent cure is generally effected.

Program for Administering Federal Aid

By **S. PERCY HOOKER**

State Superintendent of Highways of New Hampshire

WE may expect National aid in highway construction within the next four years. I take up the question as given me, therefore, without arguing as to whether the Federal Government should participate. Three broad plans present themselves as to Federal aid: 1. Shall the Federal Government adopt certain highways regardless of State lines and proceed to improve and maintain such highways irrespective of the States in which they lie? 2. Shall the Federal Government apportion certain sums of money for the care and maintenance of post roads and contribute to the town in which individual roads lie either a proportion of the cost of maintenance or a fixed sum per mile? 3. Shall the Federal Government apportion a sum of money to be divided among the different sub-divisions of the States for construction of improved highways regardless of their future maintenance?

I believe the arguments advanced for any of these three plans are based upon selfish ideas. The motor tourist realizes that through roads would be of an advantage to him and in almost every instance is an active adherent of the first plan. The farmer feels that any direct contribution from the National Government would relieve him of so much direct tax in his own town and he is for plan 3. The congressman, looking at the "pork barrel" which this appropriation would become were plan 2 adopted, feels that by log-rolling he could procure a larger share of the money within his district than in any other way.

Three Faults of Proposed Plans

If the Federal Government builds and maintains certain through highways within a State there must be created a department which would dwarf many of the other departments of Government. I think also it would tend to the standardization of roads, which, in view of the great diversity of soils and of traffic, would be the most unfortunate proposition that could be attempted. It would be the height of foolishness to build in sections of Arizona and Kansas a standard highway which would be applicable to Westchester County, N. Y., and Suffolk, Mass., or Chester, Pa., and anything that would tend to this result would be a fatal blow at the principle of Federal aid. I believe that under State aid more millions have been wasted in building too expensive roads than by committing any other error.

I do not recall that I have seen an estimate of the cost of building national highways which exceeded $250,000,000, but it is clear to my mind that this is nothing but an entering wedge and that the cost, if this plan is followed, will run into the billions. Such highways, further, cannot be equitably apportioned on any base or ratio among the States. Suppose the transcontinental route selected should run across Minnesota, Dakota, Montana, Idaho and Oregon. Does any one for an instant believe that the route through Missouri, Kansas, Colorado, Utah, Nevada and California could be omitted from the plan of through highways? Will not the advocates of the southern route—Arkansas, Oklahoma, Texas, and New Mexico—be equally as insistent? If you build a concrete highway on the middle route will you not have to duplicate on the other two?

As to maintenance, if the entire United States Army were put out to maintain and patrol the roads on the completed system there would not be enough men left to guard the coast defenses. The head of such a department would know personally nothing of the work done by his subordinates. Were he changed in office once in every four years he never would see a vast majority of the roads built under his direction.

I have nothing more to say in favor of plans 2 and 3 than of plan 1. The money sent in to the town treasurer on a rural delivery route would be used in the same manner that the town money now is upon that road. I believe that, on the whole, present conditions would not be very much improved, and while the cost would be what successive Congresses cared to make it, the improvement of the road would be practically negligible.

Program of Proposed Plan

To turn from destructive to constructive criticism, I offer a schedule which I think a reasonable method of administering Federal aid.

Organize a department of highways under the Federal Government. Insist upon the organization of a State department of highways in each State as a condition precedent to receiving any State aid. Cause road maps to be furnished by each State department of all the roads within the State showing comparative assessed valuation per mile of the State as a whole as a basis for Federal aid. These maps should show the comparative status of those highways which in the estimation of the State department might be designated as through routes. Compute the proportionate per cent which the Federal Government should contribute by using three factors—population, mileage and assessed valuation—giving the States different ratios based upon this statement. Have the State department certify to the Federal department the amount which it would have available for construction and maintenance in any given year and base the Federal aid upon the amount raised apportioned from the ratio previously found.

Have the State department file with the Federal department an application for aid upon given roads, indicating them upon the maps. These roads should be either continuous highways to be built under Federal aid or connecting highways already built under State aid forming continuous highways. Have the State department file its plan indicating the class or type of road which it desires to build. Have the Federal department pass upon the general plans of the State department for the various types of road. Require the consent of the Federal department to the type indicated as being used. Have the entire work of construction and inspection in charge of the State highway department. Have finished work inspected by the Federal department and accepted or refused as it complies with the specifications for the class selected. Have the State highway departments certify as to the actual cost of the improvement with such detail as the Federal Government might insist upon and then have the latter remit such proportion of the cost as had been apportioned before the work was begun. Have the money given by the Federal Government for maintenance expended only upon such highways as had been built under Federal aid, and in the same proportion as the ratio of cost. Give to the Federal department the right to withhold all aid both for construction and maintenance if the highways built with its aid are not properly maintained.

I realize that this is simply the plan of State aid as adopted by many of the States, except that the supervision is placed not with the Federal but with the State highway department. The reasons for this are that the State department would be far more likely to be conversant with the needs, wishes and desires of the travelers than the Federal department, and with the clause that such Federal aid should be expended only on continuous highways a system would be completed far more rapidly. It seems to me that by this plan the State departments would be able to select the class of construction best adapted to local conditions and that, if Colorado or New Mexico were satisfied with a graded highway, while New York and Massachusetts required a pavement, the best interest for all would be conserved by permitting the building of the class of road which the State officials thought satisfactory.

The Federal Government having made specifications as to how a pavement should be laid, should be satisfied to award Federal aid, provided these specifications were in general carried out and there was no difference of opinion as to how much per mile should be expended.

Designation of Maintenance Funds

My first thought was that the sum which the Federal Government made available for each State should be used where needed either in maintenance or construction. I have decided, however, that in many instances such a course would result in not extending the construction of new highways, and my plan limits the maintenance given by Federal aid to stretches of highway which have been constructed under Federal aid.

Perhaps the most difficult matter of adjustment according to this plan would be the proportionate amounts which the different States should receive. There can be no question that the State with a large road mileage and small assessed valuation should receive a larger proportion of Federal aid than the richer State, and the first idea is that naturally the assessed valuation per mile should be the unit in determining a proportionate aid; but it seems to me that population should also be taken into account.

In many of the States the laws have been amended so that a considerable portion of the State aid is now spent within villages. This I consider unwise. I realize that on through highways frequently the outlet to such highways, even in the cities, is in far worse condition than the country road, but it does not seem that even State-aid money should be spent upon improving the streets within a municipality.

Problems of Administration

By Col. EDMUND A. STEVENS

State Highway Commissioner of New Jersey

THE condition of the public roads to-day is generally admitted as a good measure of a community's advance in civilization. This condition depends upon two factors—first, correct planning, design and construction, and, second, proper upkeep. Both factors are in turn dependent on efficient administration, including all of the numerous details of management, exclusive of the technical engineering work. As a pioneer nation, we still act on the principle that any two-handed man who can earn a living and is fairly honest can care for a road and that the highway engineer is an almost superfluous luxury whose main duty is drawing pretty but useless pictures of work and whose service and pay should be cut down to the minimum.

Trained Engineers Essential

The engineer is essential to good roads; you can not build them and have them stay built without his aid. We too often spend State money in building magnificent roads without taking any thought of their administration after completion. In New Jersey, as in many other States, we turn over these roads to elective bodies of constantly changing personnel and politics—bodies not fitted by training for the technical work of administration—without requiring the employment of trained men and without setting any required standard of maintenance. At best these bodies are unconnected units and their work is not systematized and co-ordinated to a common end.

Whatever may have been the case twenty years ago, the motor vehicle and its ever-growing commercial importance to-day demand continuous roads. The condition of every part of such a road is a matter of general concern and not to be localized if the value of the investment is to be realized. While this statement is true as to all public roads, its importance is greatest in the case of the main arteries of traffic. These will naturally cross municipal and county boundaries. The cost of the upkeep of any portion within one municipality bears no close relation to the benefit to the community nor to its financial ability. In the case of less important roads the benefit accrues largely to the locality. Hence it becomes advisable to classify roads so as to provide equitably for the distribution of the cost of maintenance.

In whatever manner this classification is made and whatever system of administration is chosen, certain underlying principles must be adopted if the best results are to be expected—there must be a central control to secure the direction of all efforts to a common end, an adequately trained force, and sufficient funds.

As to the first of these principles, whether there be a system of State highways or whether localities are left in charge of their own roads, someone must lay out the through lines and there must be some uniformity in their design and improvement as well as in the condition in which they are to be maintained. These cannot be left to the chance agreement of a number of municipalities. Work both in construction and repair must be planned to give the best results for the State at large. For the purposes of scientific design statistics as to traffic over various types of roads and the comparative cost of the maintenance under varying conditions of load must be gathered, compiled and analyzed. New types of construction must be tested and proved. All of this is work which local bodies are seldom fitted to perform, even were their means and the value to them of the data obtained sufficient to justify the considerable expense involved.

Organization

The need of a trained and disciplined force is beyond doubt. The most important men will be those at its head and these naturally will be engineers. Highway engineering in its theoretical requirements is simple enough. Except as to bridges few problems arise requiring high mathematical training. There is, however, an almost unlimited field for the application of the thoroughly digested information that comes only with extended experience. The power to acquire, digest and systematically apply this information is what makes the practical engineer, such as is needed in highway work. This training is best acquired on the work itself by young men with thorough technical preparation. The ideal road force must attract such men, must offer them a training and a future. Only in a large force, free from political control and with fair chances for promotion as reward for proved efficiency, can such conditions be insured.

That which is true for the technically trained young man is likewise true for the repair man. All officials actually in charge of roads, no matter how important or unimportant their tasks, should form units of a trained force, without consideration of whether they be employed by township, county or State. To every State official will, I believe, at once occur many instances of waste by improper repair, of petty graft and of stupid or ill-planned work. Without a force which at least is supposed to be trained, it is hard, if not impossible, to enforce that individual responsibility which must underlie any efficient administration. Good roads are needed under all jurisdictions if the commercial potentiality of any general highway system is to be developed; hence the co-ordination and unity not only in the State forces but in all officials charged with providing efficient road service for the people of any State. To-day the civil service system seems the only means of recruiting such a force. Whatever may be the shortcomings of this source of supply, it is preferable to political control. Above all, any force must have a spirit of loyalty, of devotion to their work and of pride therein; in a word, an "esprit de corps." With this, men far from ideal in attainments and organization can and will accomplish results impossible to the most perfectly fitted and organized body not animated by this life-giving force.

Methods of Administering Funds

In providing for original improvement, the choice of the bond-issue or of the "pay-as-you-go" method or some combination must be made. The elements in this choice are the comparative cost to the taxpayer and the return to the public in road service. My own experience is that these are seldom considered. The decision is usually based on the desire of finding the easiest way out of a difficult dilemma, satisfying the demand for good roads and keeping down the next year's tax levy. The troubles of the officials in charge thirty years hence rarely worry one very much.

I have worked out an example as follows: Take a case involving $10,000,000 in bonds, issued at the rate of $1,000,000 a year for ten years, bearing 4 per cent interest and due thirty years from date of issue, with a sinking-fund charge of 1.783 per cent; assume that 100 miles of road will be built each year, that the contractor will keep the road in repair for one year after acceptance and that the cost or repair thereafter is $500 a mile; the last bonds will be paid in forty years. The cost to the community will be: Sinking fund (construction), $5,527,300; interest, $12,400,000; upkeep, $16,750,000; total, $34,677,300, or nearly $862,000 a year as an average. The yearly charge varies from $57,830 in the first year to $1,073,800 for the thirteenth year, at which maximum it remains to the thirtieth year, after which it decreases to the fortieth year, where it becomes $500,000, the repair charge.

If, on the other hand, a fund of $860,000 be set aside each year to keep roads built in repair and the balance used to build roads, the 1000-mile system would be finished in less than seventeen years. The cost of the system with its maintenance to the end of the forty-year period would be $25,779,000. There is a saving of almost $9,000,000 against which the community has enjoyed the use of the whole system for about nine years' more time.

The money spent will have been disbursed as follows: For construction, $10,000,000; for upkeep (forty-year period), $15,779,000; total, $25,779,000.

If the average yearly charge of $860,000 be kept up for the whole period of forty years, it would be possible to increase the total mileage to about 1525, but from that date on the upkeep charges would be $763,000 instead of $500,000.

All of these plans, as well as one providing for a gradually increasing charge, should receive consideration before deciding on the method to be followed in any case.

The general consideration of how the roads, after being planned and built, are to be kept in order is a vital part of administrative work. A thorough comparison of the total cost during a period under different financial plans involves such consideration. The changes of the last few years in the character of road traffic, its unforeseen increase in amount and in destructiveness to road surfaces emphasize the need for the most liberal allowances for this item. There are certain items in the cost of a road that are permanent investments. The right-of-way is pre-eminently of this class; the grading, drainage, and, in some cases, the foundation, are of such a nature that with reasonable care they can be kept, at a small yearly charge, in such condition that no depreciation takes place. The surface, however, is frequently of a material that, after a term of years, will have to be replaced and hence an item of drainage that cannot be made good every year will introduce a depreciation charge to further complicate the problem.

However costly a good road may be in its building and upkeep, it is less costly to the community at large than a bad road. It is the duty of the road administrator to see that the road is provided and kept in a condition to return a profit to those who pay for it.

Letters to the Editor

Stress Analysis of Circular Tubes

Sir: In the Engineering Record of June 14, page 669, appears an article on "Stress Analysis of Circular Tubes," by Mr. Björgulf Haukelid, dealing with various kinds of concentrated as well as uniformly distributed loads. The theory which Mr. Haukelid makes use of in his calculations can be found in various text books, the most complete of which are Müller-Breslau's "Die Neueren Methoden der Festigkeitslehre" and Bach's "Elasticität und Festigkeit." The formulæ given in these works are only applicable to materials obeying Hooke's law—i. e., steel, and, to some extent, iron. Exceptions to this law are cast-iron and concrete, leather, stone and metals in general. The writer, therefore, wishes to emphasize that a direct application of Mr. Haukelid's equations to cast-iron, plain or reinforced concrete tunnels or tubes is prohibitive. These facts have not been pointed out in Mr. Haukelid's article, which gives the impression of being of general application. In contrast to this neglect stand his statements of the fundamental theory on which he has based his calculations. He states, unfortunately, the theory erroneously in the fundamental equations, but avoids most of his first mistakes in the application. The writer, therefore, wishes in the following to make a few corrections, and will also show the limitations of the theory of which Mr. Haukelid makes use.

In order to establish the equations for curved beams it will be necessary to refer briefly to the fundamental theory as given in Bach's "Elasticität und Festigkeit." The theory, as far as it concerns this article, may be stated briefly as follows: In any curved beam, whose center line is a plane curve, supporting a load P at some point there exists at any cross-section of the beam a normal force and a bending moment, due to which the neutral axis does not coincide with the center line. The bending moment is taken with a positive sign when tending to decrease the radius of curvature of the center line, and if the radius tends to increase the negative sign is used. The normal force is reckoned positive when tensile and negative when compressive.

From Fig. 1 the following equations may therefore be established:

$$S = \frac{N}{F} + \frac{M}{Fr} + \frac{My}{\varkappa Fr(y+r)} \qquad (1)$$

where S is the unit stress at any point $\pm y$ length units from the center line, N the normal stress, M the bending moment, F the area of the cross-section, r the radius and

$$\varkappa = -\frac{1}{F}\int \frac{y}{r+y}\,dF \qquad (2)$$

Looking at Fig. 1 it is seen that the original radius, through the influence of the forces, assumes the value ϱ. The angle undergoes a similar change from $d\varphi$ to $d\varphi + \Delta d\varphi$. Calling

$\Delta d\varphi / d\varphi = \omega$, the total change of an angle φ may be stated

$$\Sigma \Delta d\varphi = \int \omega d\varphi \qquad (3)$$

The value of ω can be shown to be

$$\omega = \frac{1}{EF}\left(N + \frac{M}{r} + \frac{M}{\varkappa r}\right) \qquad (4)$$

The above equations (1), (2), (3) and (4) are applicable to any beam whatsoever, so long as the following conditions are fulfilled:

(a) The beam must be curved in one plane only;

(b) The beam must be continuous and of constant cross-section;

(c) The stresses in the beam must follow Hooke's law.

(d) The temperature must be uniform throughout the beam.

Returning to Fig. 1, it will be noticed that y may have any value from zero to e. If, now, e is small in comparison to the radius r the value of

$$\varkappa = I/Fr^2 \text{ approximately}$$

I is the moment of inertia of the beam section. Introducing this approximation, our formulæ assume the following appearance:

$$s = N/F + M/Fr + Myr/I(r+y) \qquad (1a)$$
$$\omega = (1/EF)(N + M/r + MFr/I) \qquad (4a)$$

Now, in order to reach Mr. Haukelid's plane, we must make still another approximation—i.e., M/r will have to be considered as of no consequence. That this in some cases is a permissible step becomes clear when viewing equation (4a), where the term M/r decreases while MFr/I, the last term, increases when r increases. Equation (4a) may therefore be written

$$\omega = \frac{1}{EF}\left(N + \frac{MFr}{I}\right) \qquad (4b)$$

Now, if the reader will number Mr. Haukelid's equations by (1), (2), (3), etc., the writer will point out where Mr. Haukelid has gone astray. Mr. Haukelid says in the middle of the first column of his article: "In order to determine M_A we note that the total change of angle from A to B is zero. In other words, the angle AB always remains 90 deg. or $\int_0^{\frac{\pi}{2}} d\varphi = 0$". It is easy to see that he confuses the constant angle AB with the variable angle φ. The equation he is driving at is evidently equation (3) of this article, or $\Sigma \Delta d\varphi = \int \omega d\varphi$, which, if integrated between the limits 0 and $\pi/2$, becomes $\int_0^{\frac{\pi}{2}} \omega d\varphi = 0$. But even this form of the equation is of no value unless the two concentrated loads P (see Mr. Haukelid's Fig. 1) are located at B, thus in reality being one concentrated load. That this is obviously so will be understood if one tries to write equations for the variable bending moment between A and B. Mr. Haukelid himself gets two equations. The mathematical continuity of an equation for the moment either from B to where P is applied or from A to P ceases at P. For this very reason we cannot integrate the last equation between 0 and $\pi/2$, as Mr. Haukelid has stated. The correct equation as applied to Figs. 1, 2 and 3 of his article should therefore read

$$\int_a^{\frac{\pi}{2}} \omega' d\varphi' + \int_0^a \omega d\varphi = 0 \qquad (5)$$

And this is to be substituted for Mr. Haukelid's first equation. Proceeding further in his article we come to a statement: "But from the equation of the elastic line

$$d\varphi = Mds/EI + Nd\varphi/EF."$$

That this should be the equation of the elastic line is no doubt Mr. Haukelid's own invention, and the writer shall therefore leave this statement without comment and instead present what he thinks Mr. Haukelid really means.

Looking at equation (4b) of this article we find that it can be written in this form:

$$\omega = Mr/EI + N/EF.$$

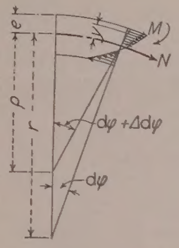
Multiplying both sides by $d\varphi$ we get

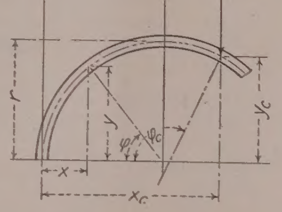
$$\omega d\varphi = Mrd\varphi/EI + Nd\varphi/EF. \qquad (6)$$

That this is the equation Mr. Haukelid is driving at there can be little doubt. His statement that $ds = rd\varphi$, since he previously said that

$$\int_0^{\frac{\pi}{2}} d\varphi = 0, \text{ is not without interest.}$$

Arriving at Mr. Haukelid's third equation we note the same tendency to integrate a discontinuous function; and, although in the application no use is made of this third equation in that form, but still another approximation made, the correct equation ought to be given. If equation (4) of this article be integrated as indicated by equation (5), Mr. Haukelid's third equation takes the form





$$\int_a^{\frac{\pi}{2}}\left(\frac{M'rd\varphi'}{EI} + \frac{N'd\varphi'}{EF}\right) + \int_0^a\left(\frac{M''rd\varphi''}{EI} + \frac{N''d\varphi''}{EF}\right) = 0 \qquad (7)$$

M', M'', N' and N'' are introduced to emphasize that M and N are not the same functions from 0 to $\pi/2$, but discontinue at P. We shall now proceed to apply these approximate formulæ to Case 1 of Mr. Haukelid's article.

Before we can apply equation (7) the bending moments M' and M'', and also the normal forces N' and N'' must be determined. (See Fig. 2 of Mr. Haukelid's article.) At point A the bending moment is evidently $+M$, since it tends to bend the "beam" still more Besides the bending moment there also exists a normal force equal to $-P$. Now consider a section at C where the bending moment is M', the normal force N', and the subtending angle φ. From the figure it is seen that

$$M' = M_A - Pr(1 - \sin\varphi),$$

and

$$N' = -P \sin\varphi$$

Similarly the bending moment between α and o is

$$M'' = M_A - Pr(1 - \sin\alpha)$$

It is evident that N'' is zero.

The values here found from M', M'', N' may now be substituted in equation (7). To follow Mr. Haukelid's example we shall drop the terms $Nd\varphi/EF$, so as to get a result which will allow of comparison with that of Mr. Haukelid. This will therefore be the third approximation of the original equations. Equation (7) therefore takes the new form

$$\int_a^{\frac{\pi}{2}}\{M_A - Pr(1 - \sin\varphi)\}\,d\varphi + \int_0^a\{M_A - Pr(1 - \sin\alpha)\}\,d\alpha = 0.$$

Thus

$$M_A = 2[Pr(\pi/2) - Pr\cos\alpha - Pr\alpha\sin\alpha] \div \pi$$

By putting $r \sin\alpha = a$, as Mr. Haukelid has done, and then writing the equation his way, we get

$$M_A = -2[Pa\alpha + Pr\cos\alpha - Pr(\pi/2)] \div \pi$$

This is the result Mr. Haukelid arrived at, the only difference being in the sign. To show within what limits the formulæ presented in this article may be used in their approximate form, it will be necessary to go back to equation 2.

There

$$\varkappa = -\frac{1}{F}\int \frac{y}{r+y} dF$$

By multiplying this equation by Fr we get

$$Fr\varkappa = -r\int \frac{y}{r+y} dF$$

By adding $\int ydF = 0$ to the right-hand side. enables us to write the equation thus:

$$Fr\varkappa = \int \frac{y^2}{r+y} dF$$

Again multiply by r on both sides and transpose the expression under the integral sign, and we have

$$Fr^2\varkappa = \int \frac{y^2}{1 + y/r} dF$$

Now it is easy to see that the value the integral approaches when r increases indefinitely is $\int y^2 dF$. But this is the expression for the moment of inertia about the center line, hence

$$Fr^2\varkappa = I \text{ when } r = \infty$$

and

$$\varkappa = I/Fr^2.$$

Now the fact is that this value of $\varkappa$ may safely be used even for quite limited values of r, as the following tables will show.

According to Professor Bach's "Elasticität und Festigkeit," page 491, the following discrepancies occur if I instead of $Fr^2\varkappa$ is used.

When

$r =$	h	$2h$	$3h$	$4h$
$Fr^2\varkappa/I =$	1.18	1.04	1.02	1.01

I is here the moment of inertia of a rectangular section where h is the height. Thus $I = bh^3/12$ if b is the width of the section.

From the above table we notice that the error is only 1 per cent when $r = 4h$. If the section of which I is the moment of inertia is other than a rectangle, the error will not be much more than 1 or 2 per cent so long as $r = 4h$. We may, therefore, safely use the approximate formulæ when $r \geq 4h$.

But now we come to the most important step in our calculations, viz., that of determining the safe maximum stress. This side of the problem is entirely neglected in Mr. Haukelid's article. The writer therefore presumes that not a few, after having found the expression for the maximum bending moment M, would simply calculate the maximum stress according to the formula for straight beams thus: $S = Me/I$, instead of using the formula for curved beams:

$$S = \frac{P}{F} + \frac{M}{Fr} + \frac{Myr}{I\,(r + y)}$$

which in the beginning of this article was numbered (1a). In order to explain the difference we shall take a concrete example. Let $M = M_A = -\,2[Pa\alpha + Pr\cos\alpha - Pr(\pi/2)] \div \pi$; $P = -\,10{,}000$ lb.; $\alpha = 0$; $r = 100$ in.; $y = \pm e = \pm h/2$, and $r = 4h$. Thus $r = 8e$ and $e = 12.5$ in.

Substituting the values of P, r and α in the first equation given for M_A makes

$$M_A = 364{,}000 \text{ in.-lb.}$$

Now calculate the moment of inertia, etc., from equation

$$S = Me/I$$

Assuming S = 16,000 lb. per square inch:

$$I = 364{,}000 \times 12.5/16{,}000 = 284.4 \text{ in.}^4$$

The area of the section is found by putting the width of the section = b, and then determine b thus:

$$b(12.5)^3/12 = 284.4 \text{ or } b = 1.747 \text{ in.}$$

Hence the area $F = 25 \times 1.747 = 43.7$ sq. in.

Now substitute these values in the equation

$$S = \frac{P}{r} + \frac{M_A}{Fr} + \frac{M_A yr}{I\,(r + y)}$$

and put $y = -\,e$ so as to get the maximum stress, i.e., compression. The compressive maximum stress at point A is therefore

$$s = -\frac{10{,}000}{43.7} + \frac{364{,}000}{43.7 \times 100} - \frac{364{,}000 \times 100 \times 12.5}{284.4\,(100 - 12.5)}$$

and $S = -\,18{,}430$ lb. per square inch.

This shows a discrepancy of

$$(18{,}430 - 16{,}000)/18{,}430 = 13 \text{ per cent.}$$

Considering now that we have been using approximate formulæ for determining M_A, the actual discrepancy may be 20 per cent or more, instead of 13 per cent. And, furthermore, if the radius r should happen to be less than $4h$ these approximate formulæ may be out 30, 40 or 50 per cent, the greatest possible error being about 75 per cent.

The next two of Mr. Haukelid's fundamental equations are those for the elongation of the vertical and horizontal diameters. Instead of criticising the merits or demerits of these formulæ, which would only take up valuable space, the writer wishes to present a slightly approximated form of the correct formulæ as given by Professor Bach. Using Mr. Haukelid's notation as far as it suffices, and for the rest referring to Fig. 2 of this article, we get

$$\Delta x_c = y_o \int_o^{\varphi_c} \frac{Mr}{EI}\,d\varphi - \int_o^{\varphi_c} y\,\frac{Mr}{EI}\,d\varphi + \int_o^{x_c} \frac{N}{EF}\,dx$$

and

$$\Delta y_c = -\,x_o \int_o^{\varphi_c} \frac{Mr}{EI}\,d\varphi + \int_o^{\varphi_c} x\,\frac{Mr}{EI}\,d\varphi + \int_o^{y_c} \frac{N}{EF}\,dy$$

From these equations it is comparatively easy to check Mr. Haukelid's formulæ and his tabulated results.

O. G. WELLTON,
Assistant Professor of Mechanical
Engineering, Queen's University.
Kingston, Ont.

Sir: Prof. O. G. Wellton's criticism of my article in the Engineering Record of June 14, page 669, has been submitted to me for a reply. In answer to his statement that my formulæ cannot be used for cast iron and concrete I have to say this:

It is the general practice, and recognized as correct, to use Hook's law and constant modulus of elasticity for both cast iron and concrete within the limits used for allowable unit stresses. Tests confirming the correctness of this statement have been made here and abroad, and for the purpose of answering Professor Wellton I will only refer to those made in the United States. The records of these tests are to be found in the University of Illinois Bulletin 22 under the title "Tests of Cast-Iron and Reinforced-Concrete Culvert Pipe," by Prof. A. N. Talbot. On page 60, under "Comparison of Methods of Loading," there is the following statement: "The tests under concentrated loads for both the cast-iron and the reinforced-concrete rings gave results which are consistent with analyses, both as to strength of the rings and the nature of the deflection of the curves and the amount of deflection."

On page 64 of the bulletin it is further stated: "The plain concrete rings broke under slight deflections at loads agreeing well with the calculated strength both for concentrated and distributed loads."

In spite of Professor Wellton's assertion that the use of the formulæ is prohibitive for cast iron and concrete I believe that within practical limits they are correct for both materials, and I think that engineers will support me in this statement.

In criticizing certain details in the mathematics of my article it seems to me that Professor Wellton is splitting hairs. In my analysis I have neglected all quantities that are of no practical value; my object was to produce useful formulæ for engineers. I made no attempt at a strict mathematical treatment of the subject. Experience shows that my approximations are justified and all authorities recommend their use.

Professor Wellton says that the most important step in our calculation is the determination of safe maximum stress. Taking his own figures for load and moment, he says that if the unit stress for bending is 16,000 lb. per square inch, he finds that the unit stress at A is only 16,000 lb. per square inch, if the straight-beam formula is used. In the very beginning of my article it was stated that at A there is a moment plus a direct load, and Professor Wellton's neglect of the direct load is absolutely wrong.

If the load were placed to make the direct load the dominating factor and the moment small, Professor Wellton would only get a small unit stress due to the moment, while the actual unit stress would be large.

The correct unit stress at A is $S = Me/I + P/F$, according to the straight-beam formula. If we chose the unit stress at A in bending to 16,000 lb. per square inch, as is done by Professor Wellton, the actual combined unit stress is found as follows: 364,000 (12.5/16,000) = 284.4 in.^{4} and $b(25^3/12) = 284.4$ giving $b = 0.218$ in., and the area = $0.218 \times 25 = 5.46$ sq. in.

Note the mistake in Professor Wellton's figures. The correct unit stress is: 16,000 + (10,000/5.46) = 17,835 lb. per square inch.

Considering the beam curved, the unit stress is 19,440 lb. per square inch. The difference is (19,440 — 17,835) / 19,440 = 8.25 per cent, which is quite different from Professor Wellton's 13 per cent. Furthermore, Professor Wellton's statement that the actual discrepancy may be 20 per cent or more is nothing but guesswork and has to be treated as such. That the discrepancy may be 20 per cent or more is due to previous approximations. I will show how much influence one of the approximations—that of neglecting the normal force and the deformation—has on the result in figuring the moment at A.

If the normal force is considered the moment at A will be

$$M_A = -\,Pr\left(\frac{\pi - 2}{2\pi} + \frac{h^2}{12r^2\pi}\right)$$

$$= -\,(364{,}000 + 20{,}000\,\frac{25^2}{12 \times 100^2 \times 3.14}$$

$$= -\,367{,}300 \text{ in.-lb.}$$

against 364,000 in.-lb., without taking the normal force into the account. My other approximation amounts to the same thing. I did not mention anything about figuring the shell according to a curved beam formula because I assumed that engineers reading an article of this kind would be familiar with such matters.

New York. BJÖRGULF HAUKELID.

Simple Beam and Column Formulæ

Sir: The writer has used the following approximate formulæ for some time and thought they might prove useful to his fellow-readers. They are only in part original with the writer. The beam formulæ are based on an allowable fiber stress of 16000 lb. per square inch and the column formula is based on an allowable stress of $16000 - 55l/r$ (the formula given in the Bethlehem handbook).

W = safe load in thousands of pounds, uniformly distributed; w = weight of beam in pounds per linear foot; d = depth of beam in inches; s = span of beam in feet.

W for standard I-beam and Bethlehem I's $= wd/s$; w for standard channels and Bethlehem girder beam $= 1.1\,wd/s$.

The deflection, D, in inches, of a standard I-beam, channel, Bethlehem I-beam or Bethlehem girder beam under its safe uniformly distributed load $= L^2/60d$.

Twice the depth of the beam in inches (either standard I-beam channel, Bethlehem I-beam or girder beam) gives the span in feet at which the deflection is 1/360 of the span under its safe uniformly distributed load (1/360 of the span is the allowable deflection for plastered ceilings).

The weight of section, in pounds, of a Bethlehem H-column required to support a given load for a given length of column is given by the following formula, where w^1 = weight of H per linear foot in pounds; W^1 = load in pounds; l = unsupported length of column in feet; d = depth of H in inches; $w^1 = W^1 \div (4700 - 750\,l/d)$.

The above formulæ can be used without the help of any tables and give results within two or three per cent of the true values. To determine the unsupported length in feet of Bethlehem H-columns at which the ratio of $l \div r$ (radius of gyration) = 125, divide the depth of H (in inches) by 0.4.

ERNEST OWEN,
Designing Department, Riverside
Bridge Company.
Martins Ferry, Ohio.

A SEWER AT EUGENE, ORE., providing for a population of 100,000, has just been completed at a cost of about $200,000. The outfall in the Willamette River, which carries the sewage from practically the entire city, is 90 in. in diameter and is so located that it can readily be extended downstream if the city grows in that direction and the sewage becomes a nuisance. Tentative provision was also made for using the effluent on a sewage farm in case future legislation forbids discharging into the river.

After the Cave-In—Raising Car Tracks on Randolph Street

Chicago Street Caves In Opposite 200-Foot Building

Accident Occurs While Excavation for Trench, Protected by Steel Sheet Piling, Is Being Carried Down for Wall Foundations

SETTLEMENT of Randolph Street, Chicago, in front of the new 200-ft. Marshall Field addition east of the alley between State Street and Wabash Avenue occurred about 11 p. m. Dec. 2. Coincident with the drop in the pavement, which appeared to pedestrians to be caused by passing street cars, the steel sheet piling, driven on the face of the curb wall at the outer edge of the sub-sidewalk space, 16 ft. from the property line, went down a maximum of 2 ft. 9 in., bending the girders carrying the floors for the sidewalk and the basement and sub-basement. The third basement floor, 42 ft. below street level, is to be of reinforced concrete 24 in. thick, and will rest on the clay. It had not been placed, as excavation had only proceeded about 3 ft. below the sub-basement floor.

All structural steel has been erected and the building, which has a frontage of 150 ft. on Randolph Street and 120 ft. on Wabash Avenue, is practically closed in. The contractor, John Griffiths & Son Company, was rushing the work on the sidewalks so as to open them for public use.

Movement of Sheet Piling

Between 9 and 10 p. m. workmen excavating a trench along the piling under the sub-basement floor noted seepage through the sheeting and a little later water coming up from below into the trench. When the piling began to show signs of movement the men left, but the basement was not flooded, as reported, although the water, which was turned off in twenty-one minutes by the water department, filled the soil under the pavement and came through to the surface.

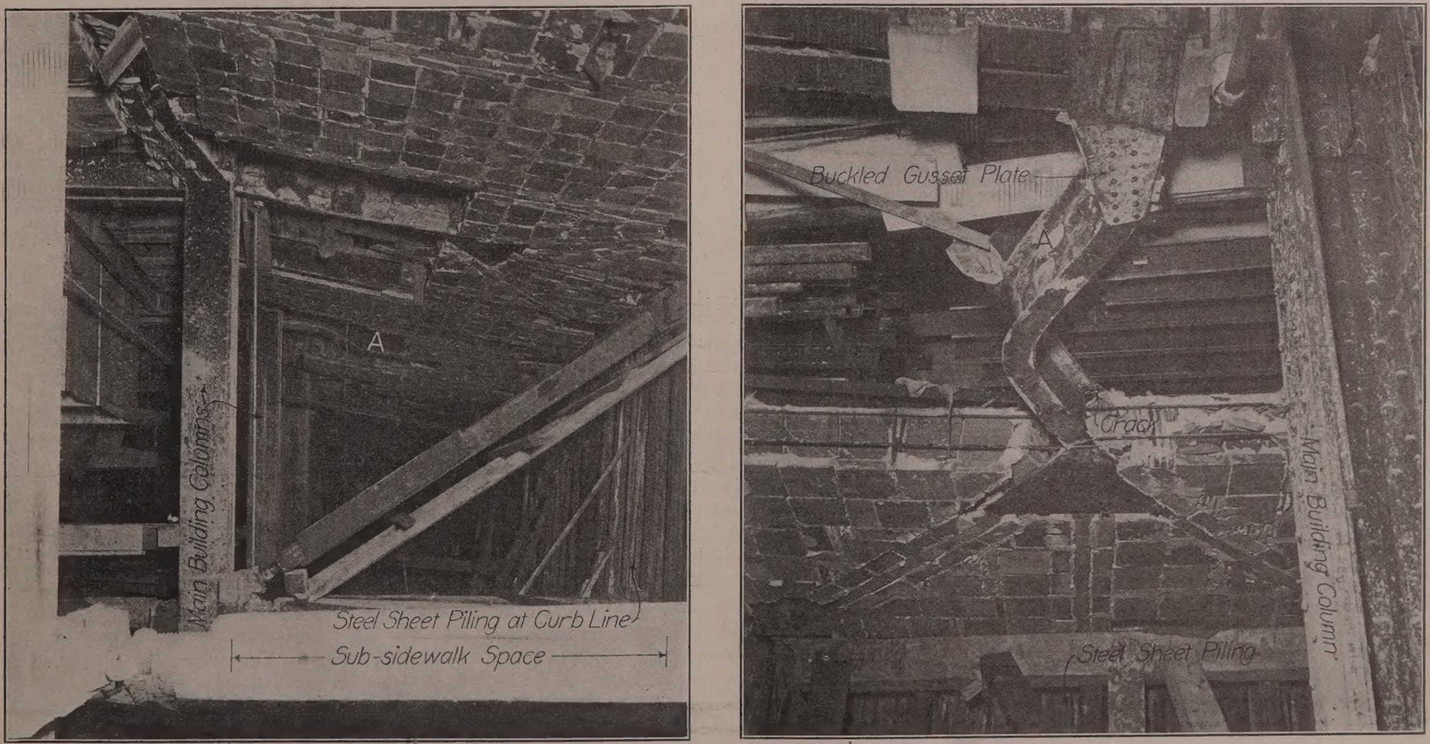
The 49-ft. United States steel sheet piling is reinforced every other pile by 12-in. I-beams. Later a concrete wall is to be built against the sheeting with a footing of sufficient width to reduce the pressure from the loading to 1½ tons per square foot. Men were excavating the trench preparatory to placing this wall, but were still 14 ft. above the bottom of the sheet piling, which extended about 5 ft. below the bottom of the lowest basement floor, according to Mr. J. G. Giaver, chief structural engineer for the architects.

Water Main Breaks

Previous to the accident the street had gone down about 6 in., not an unusual amount. On the north side of the street is a 12-in. water main from which a 6-in. connection was laid across the street to supply the former building. This 6-in. pipe was pulled out of the joint at the tee. Then, according to Mr. Giaver and Mr. E. R. Graham, supervising architect, water went down the outside of the sheeting, under it, and came up inside the excavation. This lubricated the sheeting sufficiently to permit it to drop under the floor loads it was carrying. In other words, much of the skin friction was eliminated. A chute had been constructed into the Illinois tunnel to which excavated material was being conveyed. Water found its way along and through the chute to the tunnel, the floor of which is level with that of the lowest basement. Most of the water, it is thought, escaped through the sewer.

Engineers for the gas company found a maximum depression of 5 ft. 4 in. in the street grades.

Mr. Giaver states that the sheet piling did not bend but rotated slightly, as the top is cut 6 in. and at the sub-basement floor it is in a maximum of 18 in. The inward motion of the sheet piling had therefore to "flow" a 14-ft. depth of clay. Supporting the two floors under the sidewalk are 15-in. I-beams,



Sub-Sidewalk Space, Showing Buckled Floorbeams and Distortion of I-Beam A When End Framed to Sheeting Dropped

Photographs Courtesy Chicago Daily News.

which also act as struts to take the horizontal thrust of the earth against the sheeting. There are two between columns and one at each main building column. The basement floor, the under side of which is shown in the photograph, is of tile, but the sub-basement floor is a 16-in. concrete slab in which the 15-in. I-beams were embedded. Mr. Giaver states that the concrete was too green to offer much resistance. Both of these floors outside the property line act as horizontal strut plates, transmitting the pressures back into the main building basement floors. The intermediate sub-sidewalk I-beams bent the main floorbeams between columns a maximum of 18 in. out of line. The main building columns rest on concrete caissons, which do not come above the lowest basement floor. The columns are not injured and the stability of the main structure is not questioned.

When the sheeting dropped, one of three things must have happened to the unsupported 8-ft. length of 15-in. I-beams opposite the main columns. The column pushed in an amount equal to the difference between the horizontal and diagonal dimensions; the sheeting pushed out into the street; or the I-beam shortened. The latter appears to have happened to the beam shown in the illustration. No rivets were sheared in the connections shown, but Mr. Giaver states that rivets at some of the other gusset plates were sheared.

Repairs

Repairs will consist in leaving the sheeting as it is, raising the outer ends of the I-beams supporting the sub-sidewalk floors and riveting them to the sheet piling at the new level. Those which are bent will be replaced.

A detailed investigation has been undertaken by engineers under Mr. John Ericson, city engineer. As a future precautionary measure Mr. L. E. McGann, commissioner of public works, has asked Mr. Henry Ericsson, building commissioner, to withhold permits for the excavation of deep cellars until the Department of Public Works has passed on the adequacy of provisions for the protection of the streets and public utilities.

Testing Low-Head Water Turbines

HOLYOKE tests of the horizontal low-head water turbines at the Elkhart (Ind.) plant of the Indiana & Michigan Electric Company gave maximum efficiencies of 86 per cent. On account of the more ideal setting at Holyoke the makers reduced the guaranteed per cent to 80, but by improved draft tubes and attention to other details one of the wheels under test at Elkhart gave 83 per cent efficiency. The methods of making the tests were described in a paper presented recently to the Western Society of Engineers by Mr. Lucius B. Andrus. Each of the two units has four 50-in. runners, arranged tandem. When operating under 18-ft. net head the wheels are guaranteed to develop 1400 hp. at full gate opening, using approximately 51,500 cu. ft. per minute when running at a speed of 120 r.p.m. Fortunately, the Elkhart station layout has a long rectangular tailrace in which a weir could be constructed cheaply.

In a written discussion sent by Mr. L. F. Harza, consulting engineer, of Portland, Ore., a test on a similar installation was described. Each unit consisted of six runners on a horizontal shaft, under a head of 17 ft., discharging into a curved draft tube below and operating a 1500-kw generator at 100 r.p.m. Water measurement by weirs was impracticable on account of the expense. The test was, therefore, made by thoroughly traversing the cross-section of the penstock upstream from the turbines with carefully calibrated current meters. Each test extended over several hours, and the meters were read almost continuously during this time, check observations being made 1 ft. apart vertically and 20 in. apart horizontally over the entire cross-section of the flowing water, which was 20 ft. deep and 40 ft. wide. The position of the meters was controlled by vertical wires previously installed in the penstock and spaced 20 in. apart across the entire width.

Preservative Treatment of Timber in Framing

MUCH has been written about the preservative treatment of the railroad cross-tie, but comparatively little attention has been given to the use of preservatives on structural timber. In its comprehensive report recently presented at the Montreal convention, however, the committee on preservation of timber of the American Railway Bridge and Building Association included this branch of the general subject, and offered general rules applicable to framing. These notes are taken from the report.

Whenever possible all framing of timber should be done before treatment, even to the boring of holes. On a trestle the only framing absolutely necessary is to cut the piles to the right height and apply the braces. The stringers, ties and the like can be framed and treated before going to the job, but this requires a refinement of workmanship not easily attained in this generation.

Truss bridges can be framed and treated without actual assembling, but very few wooden trusses are now built, and in any case it is probably cheaper to house them in. As some framing is necessary it is important to do as little framing as possible and properly treat such framing as has to be done. When piles are cut off, the heads should be thoroughly saturated with hot creosote, after which a good coat of hot asphalt or tar should be applied. If the cap does not entirely cover the piles, the heads should be chamfered on the sides, so the water will run off quickly and not work under the cap. Vertical bolt holes should be filled with hot creosote before bolting and those in caps should be bored before treatment.

Braces should be fitted to the piles, and in no case should the skin of the pile be cut. Piles should be selected so that all in one bent will be of approximately the same size and any large protrusions on the piles should be cut off before treatment.

No satisfactory method of treating horizontal bolt holes has been developed. It is not practical to fill the holes with creosote. The necessity for treating them at all has not been shown, but piles in water that varies in height ought perhaps to be treated. Experiments might be made by boring the holes a trifle small and burning them to size, as the charring of the wood might help. However, if the bolts are kept tight there will be little trouble. In such cases it would probably be best to spike the braces with boat spikes.

In most cases it does not pay to creosote braces and girts, as they have a reasonably long life and are easily replaced. There is also a considerable amount of framing to be done on them and this would to some extent impair the value of the creosote. A good brush treatment with creosote after framing, or better, a good soaking in a tank of hot creosote, is probably as good as a pressure treatment for girts and braces if they are of the usual small sizes.

The committee does not recommend the use of creosoted material for caps, braces, stringers and ties of open-deck trestles, but does recommend it for all parts of ballasted-deck trestles. The recommendations are general and are not intended to apply to localities where untreated material maintains its usefulness for only a very short time.

Where framed bents are used on a pile foundation, the piles should be cut off high enough to keep the sill of the bent clear of the ground (the higher the better). The question of preserving the material will depend on the cost of material and its economy in yearly cost, giving the preference to preserved material on account of the lessened delays to traffic due to reconstruction.

National Transcontinental Railway Bridges

BETWEEN Moncton and Winnipeg on the line of the National Transcontinental Railway, Canada, there are 200 bridges having an aggregate length of 11 miles and a weight of 61,000 tons of steel; they cost more than $6,000,000. Some of the most interesting structures were mentioned by Mr. R. F. Uniacke, bridge engineer for the railway company, in an address at the Canadian Railway Club.

Many of the bridges are plate-girder viaducts, among which is the bridge over Little Salmon River. It is 100 ft. high, 4,000 ft. long and contains 7,000 tons of steel. In the Bostonnais River bridge, where the deck plate girders are supported on comparatively low concrete piers, the spans are tilted out of a horizontal plane so that the girder webs are slightly inclined to the vertical and provide without shimming for the superelevation of the outer rail on a curve.

Falsework for the erection of the Bush River bridge truss spans and for the preliminary temporary support of a contractor's service track was made with the main bents capped at the proper elevation to support the chamber blocks for the trusses, and surmounted by center pony bents to carry track stringers and rails and provide for a construction track in advance of the bridge erection.

The 80-ft. deck plate-girder approach span of the Okekodaski River bridge was supported at the shore end on a concrete pier that was wrecked by transverse displacement. The subsoil was found to be unable to support the 15-ft. fill that had been retained by the abutment pier. Temporary repairs were effected by supporting the end of the span on timber cribbing until a permanent low steel trestle could be built to replace the embankment.

The Lowest Forest Fire Loss for a season ever recorded is the one practically ended now, according to the Forest Service of the U. S. Department of Agriculture. While there were about 2260 fires this year, as against 2470 last year, the areas burned over were 60,000 and 230,000 acres respectively, 780,000 acres being burned over in 1911. Forest officers believe that the comparative immunity from loss has been due partly to a favorable season, but largely to a much better organization for fire prevention than has heretofore been attained. Carelessness is still, however, the cause of most of the fires.